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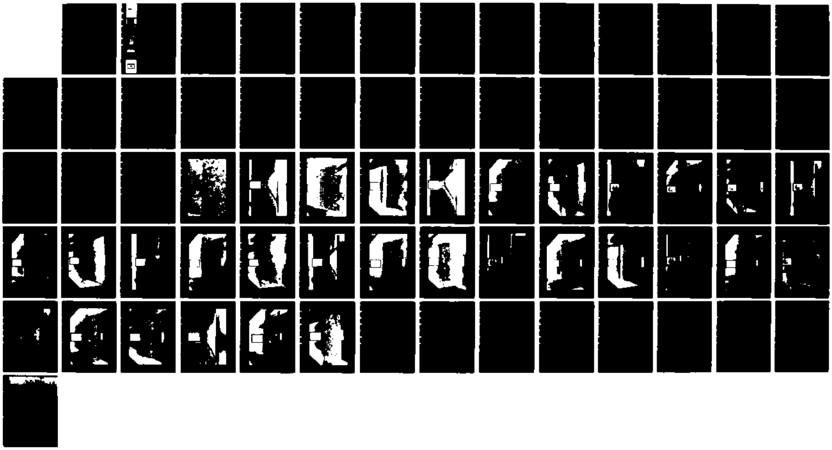

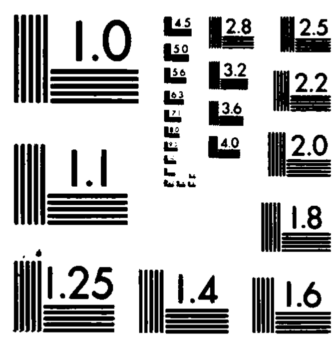
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US Army Corps of Engineers

TECHNICAL REPORT HL-83-21

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NAWILIWILI BREAKWATER STABILITY STUDY NAWILIWILI HARBOR, KAUAI, HAWAII

Hydraulic Model Investigation

by

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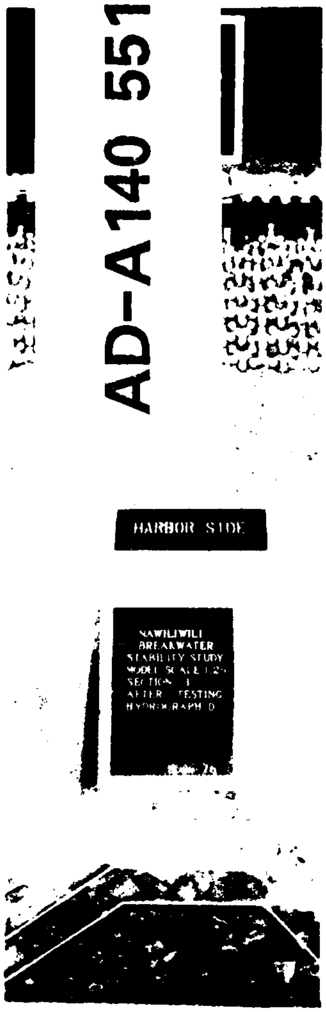

US ARMY ENGINEER WATERWAYS EXPERIMENT STATION

September 1983

Final Report

Approved For Public Release Distribution Unlimited

Prepared for U. S. Army Engineer Division, Pacific Ocean
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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

A hydraulic model investigation was conducted at geometrically undistorted, linear scales of 1:31 and 1:25, model to prototype, to evaluate the stability against wave attack of three areas of the existing breakwater at Nawiliwili Harbor, Kauai, Hawaii. Where the existing crown and/or harbor-side slope proved to be unstable, additional tests were conducted to check the stability of the rehabilitation designs proposed by the Pacific

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20. ABSTRACT (Continued).

Ocean Division. All existing and proposed rehabilitation designs were tested for the worst breaking wave conditions that could be produced for the selected wave periods, water depths, and bathymetry seaward of the test sections.

The existing, 22,000-lb dolosse on the sea-side slope at sta 19+50 proved to be unstable for the wave conditions of Hydrograph I (maximum wave height of 24.5 ft), while the remainder of the breakwater cross section (concrete crown cap and 16,000- to 20,000-lb stone on harbor-side slope) proved to be stable for the selected test conditions. The stability of the existing breakwater cross section at sta 14+00 proved to be dependent upon how tight a keyed and fitted construction of the 16,000- to 20,000-lb armor stone actually exists on the crown and harbor-side slope. Model tests results using Hydrograph II (maximum wave height of 22.5 ft) indicated that if a tight, keyed and fitted construction exists, the breakwater at sta 14+00 is an adequate design, but if this tight construction does not exist, the crown and harbor-side slope could sustain severe damage and this damage could result in the undermining and displacement of existing dolosse (22,000 lb) along the breakwater crown. A concrete rib cap and one layer of uniformly placed, 13,000-lb tribars proved to be adequate rehabilitation designs for the crown and harbor-side slope, respectively, at sta 14+00. The existing 14,000- to 24,000-lb random-placed armor stone on the sea-side slope, and 16,000- to 20,000-lb keyed and fitted armor stone on the crown and harbor-side slope at sta 10+00 proved to be adequate designs for the selected test conditions of Hydrograph III (maximum wave height of 11.6 ft).

PREFACE

The model investigation reported herein was initially requested by the U. S. Army Engineer Division, Pacific Ocean (POD), in a letter to the U. S. Army Engineer Waterways Experiment Station (WES) dated 9 July 1982. Funding authorizations by POD were granted in POD Intra-Army Order E86820010 dated 27 August 1982 and its Change Orders No. 1 and No. 2, dated 3 December 1982 and 15 April 1983, respectively.

Model tests were conducted at WES during the period September 1982 through April 1983 under the general direction of Messrs H. B. Simmons, Chief of the Hydraulics Laboratory, C. E. Chatham, Jr., Acting Chief of the Wave Dynamics Division, and D. D. Davidson, Chief of the Wave Research Branch. Tests were conducted by Mr. C. R. Herrington, Engineering Technician, assisted by Mrs. B. J. Wright, Engineering Technician, under the supervision of Mr. D. G. Markle, Research Hydraulic Engineer. This report was prepared by Messrs. Markle and Herrington.

Liaison was maintained with POD during the course of this study by means of conferences, progress reports, and telephone conversations.

Commander and Director of WES during the conduct of this study and preparation and publication of this report was COL Tilford C. Creel, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
miles (U. S. nautical)	1.852	kilometres
pounds (force)	4.448222	newtons
pounds (force) per cubic foot	157.087467	newtons per cubic metre
tons (force)	8896.444	newtons

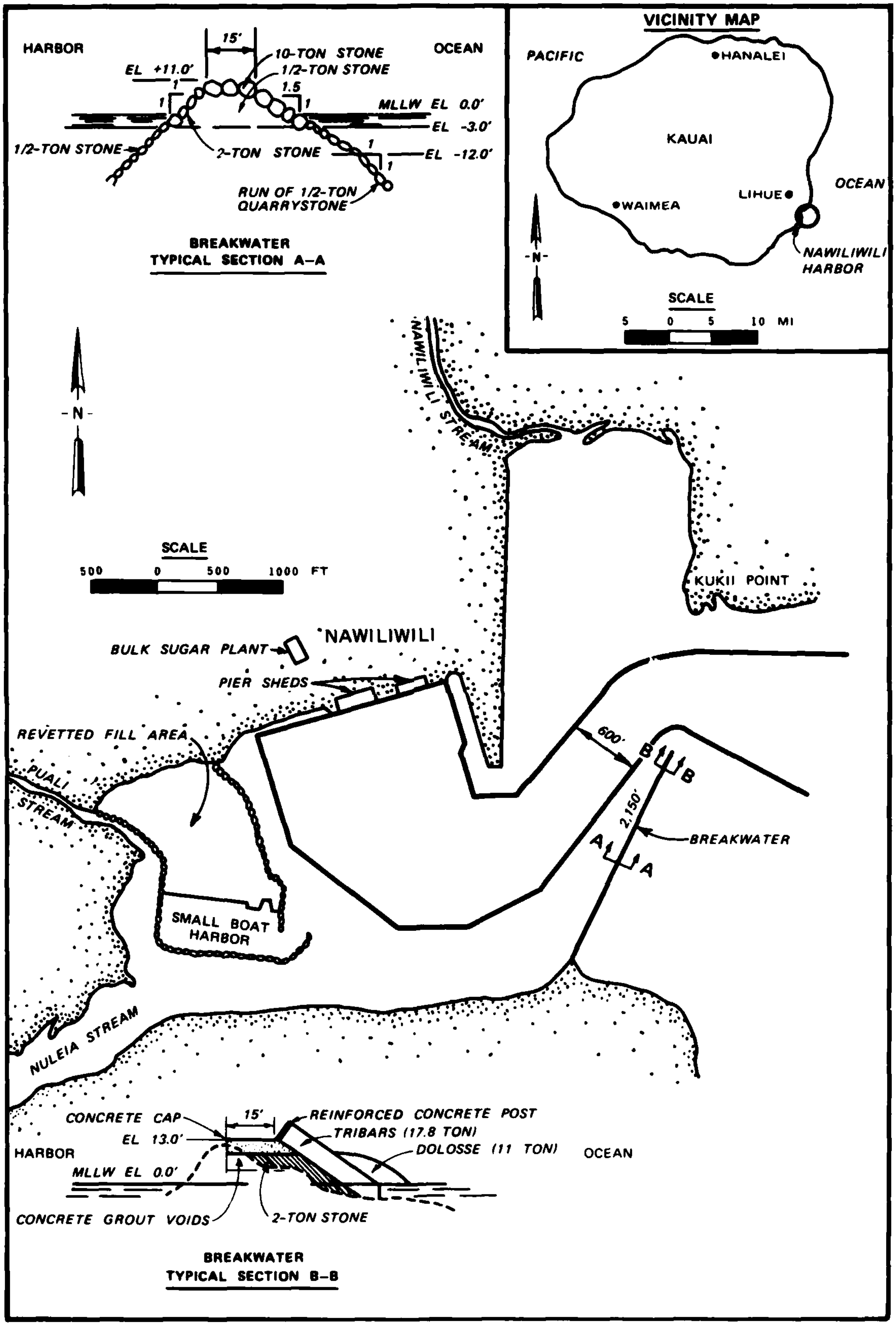

Figure 1. Nawiliwili Harbor, Kauai, Hawaii

NAWILIWILI BREAKWATER STABILITY STUDY

NAWILIWILI HARBOR, KAUAI, HAWAII

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. Nawiliwili Harbor is located on the southeast coast of the Island of Kauai (Figure 1), about 100 nautical miles* northwest of Honolulu, Oahu, Hawaii. The harbor is protected by a 2,150-ft rubble-mound breakwater. The original structure was completed in 1930 (Markle and Davidson, in preparation). Severe storms in 1954, 1956, and 1957 severely damaged the armor-stone breakwater and model tests were conducted at the U. S. Army Engineer Waterways Experiment Station (WES) in 1958 (Jackson, Hudson, and Housley 1960) to determine the best method of rebuilding the head and strengthening about 500 ft of the seaward end of the breakwater. In 1959, the head and seaward 500 ft of the sea-side slope of the trunk were rehabilitated with 17.8-ton tribars and a concrete cap was poured on the crest of the breakwater. Of the 598 tribars placed, 351 were reinforced. One layer of tribars was uniformly placed on the trunk while two layers of random-placed tribars were used on the sea-side slope of the head. A survey of the breakwater in 1975 found major deterioration of about 1,000 ft of the armor-stone trunk and several slumped areas in the uniformly placed tribars. Further inspection revealed that several of the tribar units were broken (approximately 98) and at that time model tests were initiated at WES (Davidson 1978) to determine the best method of rehabilitating the structure. The rehabilitation work was completed in October 1977. The one layer of uniformly placed tribars was overlaid with two layers of 11-ton, unreinforced dolosse (485 dolosse). The dolos coverage extended from the toe of the sea-side slope to approximately +5.0 ft mean lower low water (mllw).** For 300 ft shoreward of the tribar area, the sea-side slope of the

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

** For convenience, symbols and unusual abbreviations are listed and defined in the Notation (Appendix A).

trunk was rehabilitated with two layers of random-placed, 11-ton dolosse; 449 dolosse were placed in this area from the toe to the crown of the structure. Shoreward of the 11-ton dolosse, the sea side of the trunk was rehabilitated with random-placed 14,000- to 24,000-lb armor stone.

2. To date, the majority of the breakwater repair work has been concentrated on the head, crown, and sea-side slopes. The harbor-side slopes have degraded to very steep slopes; and the U. S. Army Engineer Division, Pacific Ocean (POD), is concerned about their stability and is considering rehabilitation of various lengths of the crown and harbor-side slopes.

Purpose of Model Study

3. The purposes of this breakwater stability study were as follows:

a. Evaluate the overall stability of the existing breakwater sections located at sta 19+50, 14+00, and 10+00.

b. If the crown and/or harbor-side slope of an existing section proves to be unstable for the selected wave and still-water level conditions, evaluate the overall stability of the respective rehabilitation design as provided by POD.

PART II: THE MODEL

Design of Model

4. Tests were conducted at undistorted linear scales of 1:31 and 1:25, model to prototype. Scale selections were based on the size of model armor units available relative to the size of prototype armor units existing on and/or proposed to be added to the prototype breakwater, elimination of stability scale effect (Hudson 1975), and capabilities of the available test flume. Based on Froude's model law (Stevens et al. 1942) and the linear scales of 1:31 and 1:24, the following model-to-prototype relations were derived. Dimensions are in terms of length (L) and time (T).

Characteristic	Dimension	Model-Prototype Scale Relations for Model Scales of 1:31	1:25
Length	L	$L_r = 1:31$	1:25
Area	L^2	$A_r = L_r^2 = 1:961$	1:625
Volume	L^3	$V_r = L_r^3 = 1:29,791$	1:15,625
Time	T	$T_r = L_r^{1/2} = 1:5.6$	1:5

5. The specific weight of water used in the model was assumed to be 62.4 pcf and that of seawater is 64.0 pcf. Specific weights of model breakwater construction materials were not identical with their prototype counterparts. These variables were related using the following transference equation:

$$\frac{(W_a)_m}{(W_a)_p} = \frac{(\gamma_a)_m}{(\gamma_a)_p} \left(\frac{L_m}{L_p}\right)^3 \left[\frac{(S_a)_p - 1}{(S_a)_m - 1}\right]^3 \qquad (1)$$

where

subscripts m, p = model and prototype quantities, respectively

W_a = weight of an individual armor unit or stone, lb

γ_a = specific weight of an individual armor unit or stone, pcf

L_m/L_p = linear scale of the model

S_a = specific gravity of an individual armor unit or stone relative to the water in which the breakwater was constructed, i.e. $S_a = \gamma_r/\gamma_w$

γ_w = specific weight of water, pcf

Test Facilities and Equipment

6. All of the two-dimensional breakwater stability tests (incident wave crests were parallel to longitudinal axis of the breakwater) were conducted in a 5-ft wide, 4-ft deep, and 119-ft long concrete flume (Figure 2). The test facility was equipped with a vertical displacement wave generator capable of producing monochromatic waves of various periods and heights.

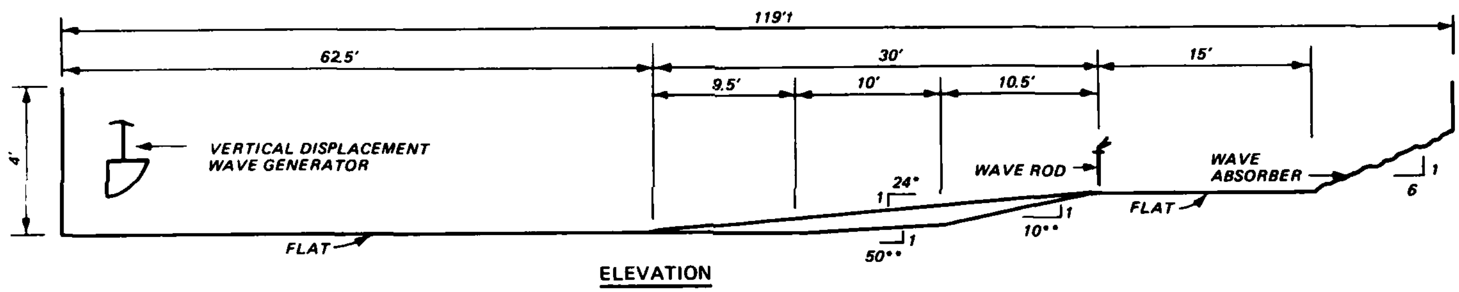
Model Construction and Testing Procedures

Modeling local bathymetry

7. Based on the results of a hydrographic survey conducted by POD, it was agreed by both WES and POD that the existing bathymetries seaward of the sea-side toe of the proposed test sections at sta 19+50 and 14+00 could be represented by a 1V-on-10H slope extending seaward of the breakwater toes and a 1V-on-50H extending seaward of the 1V-on-10H slope. The existing bathymetry seaward of the sea-side toe of the breakwater at sta 10+00 was represented in the model by a 1V-on-24H slope. Figure 2 shows the various slopes and their respective lengths as they were molded in the flume for testing the various test sections.

Flume calibration

8. Following molding of the local bathymetry and prior to installation of the first test section, the test flume was calibrated for the selected wave periods and water depths. Test waves of the required characteristics were generated by varying the frequency and amplitude of the wave generator paddle. Changes in water-surface elevations as a function of time were measured by an electrical wave-height gage and recorded on chart paper by an electrically operated oscillograph. The wave gage was located where the sea-side toe of the breakwater would be situated in the flume. This was the top of the 1V-on-10H slope for sta 19+50 and 14+00 and the top of the 1V-on-24H slope for sta 10+00.



† ALL DIMENSIONS ARE ACTUAL MODEL LENGTHS
* SLOPE USED FOR TESTING BREAKWATER CROSS SECTION AT STA 10+00
** SLOPES USED FOR TESTING BREAKWATER CROSS SECTIONS AT STA 19+50 AND STA 14+00

Figure 2. Test flume geometry and wave rod location

Methods of constructing test sections

9. Existing conditions of the prototype breakwater sections were defined by means of aerial photographs and/or line drawings furnished by POD. POD also provided line drawings of the proposed rehabilitation designs to be tested if the crown and/or harbor-side slope of an existing section proved to be unstable when exposed to the selected wave and still-water level (swl) conditions.

10. Model breakwater sections were constructed to reproduce, as closely as possible, the existing breakwater conditions and the results of the usual methods of constructing prototype structures. Core material was dumped by bucket or shovel into the flume and was smoothed to grade and compacted with hand trowels to simulate natural consolidation resulting from wave action during prototype construction. The old cover-layer stone was then added using a one layer keyed and fitted placement specified by POD. Concrete armor units used in the cover layer, or layers, were placed either in a random manner, i.e., placed in such a way that no intentional interlocking of the units was obtained, or with uniform placement, where very close spacing and some intentional interlocking of the units were achieved. (Uniform placement should not be confused with pattern placement where each unit is laid down with a predetermined orientation.) Some "special" placement of the toe dolosse was used. This toe constructon technique is described and illustrated in paragraph 16.

11. Where crown protection was provided by cast-in-place concrete caps and/or concrete ribs, it was assumed that they are or will be stable in the prototype; therefore it was not necessary that they be dynamically similar to the prototype. The model ribs and caps, constructed of Plexiglas, were geometrically similar to their prototype counterparts and were held in place in the model, thus ensuring proper transmission, reflection, and dissipation of wave energy and the assumed stability of the structure's cap.

Selection of test conditions

12. Based on anticipated prototype conditions and available prototype data, POD decided that the stability tests should consider wave periods of 12, 14, and 16 sec at an swl of +4.0 ft mllw. When the first test section for each of the three breakwater test sections was constructed in the flume, it was exposed to a range of wave heights for each of the selected wave periods. Based on model observations, the most severe breaking wave conditions with

respect to the entire breakwater section were selected for inclusion in the full length stability tests.

13. Model observations of test waves on Section 1, sta 19+50, Sections 2 and 2A, sta 14+00, and Section 3, sta 10+00 indicated that all three wave periods produced some degree of overtopping but the 16-sec waves seemed to produce the largest amount of overtopping while the 12- and 14-sec wave periods appeared to produce the most severe wave attack on the sea-side slope For this reason, all three wave periods were selected for inclusion in the various test hydrographs (Hydrograph I, sta 19+50, Plate 1 and Table 1; Hydrograph II, sta 14+00, Plate 2 and Table 2; Hydrograph III, sta 10+00, Plate 3 and Table 3).

Model operation

14. Once test conditions for the breakwater section were experimentally determined, the breakwater cover layers were rebuilt; before-test photographs were taken; the flume was flooded to the appropriate depth; and the structure was exposed to the shakedown and test wave conditions. Prototype test time was accumulated in 30-sec (model time) cycles, i.e., the wave generator was started, run for 30 sec, and then stopped. This procedure eliminated contamination of generated waves by reflections from the structure that could be re-reflected from the wave generator. After each 30-sec cycle, sufficient time was provided for the test flume to still out before the next cycle was run. During stilling time between cycles, detailed model observations of the structure's response to the previous cycle of test waves were recorded by the model operator. These observations included any movement occurring on the structure and a general statement of the condition of the structure at that point in the test. All test conditions were run for at least the durations indicated for the various test hydrographs. For instances where damage did not stabilize during the duration of a test condition, the test condition was extended until damage had stabilized or the structure was considered failed. At the conclusion of the test, the flume was drained and the after-test condition of the structure was summarized in the test notes and documented with photographs. The cover layers were rebuilt and the test was repeated. The purpose of the repeat test was to determine the presence of any uncontrolled variations in model construction technique that might affect the stability of the structure. The initial and repeat test results were very similar for all sections, except Section 2, where slightly different construction techniques were used on the

original and repeat test sections. For all tests, except Section 2, only one of the test results is reported herein. Where differences in damage occurred between two tests of a test section, the test showing the higher degree of damage was selected for reporting herein.

Methods of reporting model observations and test results

15. The following list of adjectives, in order of increasing severity, was used for recording model observations and reporting test results for each test section: (a) slight, (b) minor, (c) moderate, (d) significant, (e) major, and (f) extensive. Slight and minor were used to describe acceptable results, moderate described borderline acceptability, while significant to extensive described unacceptable conditions of increasing severity. Use of these adjectives allowed some quantification of the severity and/or amount of rocking in place, onslope displacement, offslope displacement, wave overtopping, and resulting damage accrued by the breakwater's primary cover-layer units.

PART III: DESCRIPTION OF EXISTING SECTIONS, PROPOSED REHABILITATION SECTIONS, TEST SECTIONS, AND TEST RESULTS

Sta 19+50

16. The existing prototype breakwater cross section at sta 19+50 consists of 500 to 2,000-lb core material overlaid with one layer of keyed and fitted 16,000- to 20,000-lb armor stone on a 1V-on-1.5H sea-side slope and a 1V-on-1H harbor-side slope. This original structure was constructed to a crown elevation of +11.0 ft mllw and had a crown width of 21 to 22 ft. The sea-side slope and crown were rehabilitated (1959) using one layer of uniformly placed, 35,600-lb tribars on a 1V-on-1.5H slope and a concrete cap with reinforced concrete posts, respectively. The lower sea-side slope was rehabilitated (1977) by placing two layers of randomly placed 22,000-lb dolosse from the sea-side toe to an elevation of +5.0 ft mllw. The dolos protection feathered out to one layer along the upper end of the coverage. The dolos toe was constructed using the special placement technique. Photo 1 shows a comparison of random and special placement of toe dolos units. Either during original construction or subsequent to that time, an apron of 13- to 350-lb rubble stone has built up on a 1V-on-6H slope to an elevation of -13.0 mllw on the harbor side of the breakwater.

17. Section 1, Plate 4 and Photos 2-4, was constructed in the test flume at a scale of 1:31 and reproduced, as closely as possible, the existing conditions at sta 19+50 as described in the preceding paragraph. Due to the mixing of various types of prototype concrete armor units and limited range of available model armor unit weights, it was necessary to represent the existing 22,000-lb dolosse and 35,600-lb tribars as having prototype weights of 23,318 lb and 32,995 lb, respectively. The 16,000- to 20,000-lb keyed and fitted stone was represented by average individual stone weights of 18,000 lb with a linear gradation of ±15 percent by weight. Section 1 was exposed to the wave and swl conditions of Hydrograph I and by the end of the test, the dolosse had sustained moderate to significant damage, 11 to 15 dolosse displaced upslope and 14 to 20 displaced downslope and were retained just off the dolos toe area (Photos 5-7). The dolos armor showed moderate to significant rocking in place and reorientation throughout the test. At different points during the test, it was estimated that as many as 50 dolosse showed

some type of in-place movement. Significant wave overtopping was noted throughout the hydrograph steps, but the majority of the overtopping wave energy was deflected into the harbor-side swl by the concrete cap; therefore the harbor-side armor stone was only exposed to minor amounts of overtopping wave energy. Only one armor stone was displaced off the harbor-side slope and no other movement or damage was observed. Although in-place dolos movement was noted throughout the test, no additional damage or displacement occurred during the last 45 min of Hydrograph I.

18. It should be noted that the 22,000-lb dolosse were previously designed (Davidson 1978) based on a swl of +3.5 ft mllw and a sea-side toe elevation of -15.0 ft mllw. This resulted in a water depth of 18.5 ft and a worst breaking wave height of 19.4 ft for which the 22,000-lb dolosse exhibited good stability with only minor rocking in place. By use of the Hudson equation:

$$W = \frac{\gamma_a H^3}{K_\Delta (S_a - 1)^3 \cot \alpha} \qquad (2)$$

where

W = weight of an individual armor unit, lb

γ_a = specific weight of an individual armor unit, pcf

H = design or test wave height, ft

K_Δ = stability coefficient

S_a = specific gravity of an individual armor unit relative to the water in which the breakwater is constructed, i.e. $S_a = \gamma_a/\gamma_w$

γ_w = specific weight of the water, pcf

α = angle the breakwater slope makes with the horizontal, deg

for $W = 22{,}000$ lb , $\gamma_a = 146$ pcf , $H = 19.4$ ft , $\gamma_w = 64.0$ pcf, and $\cot \alpha = 1.5$, it can be shown that this dolos stability corresponded to a stability coefficient of 15.4. Based on an October 1982 hydrographic survey conducted by POD, the sea-side toe elevation of the breakwater at sta 19+50 was defined as being -19.0 ft mllw. The swl for the test reported herein was set at +4.0 ft mllw by POD. This resulted in a testing water depth of 23 ft and a worst breaking wave height of 24.5 ft. By using Hudson's equation for $H = 24.5$ and $K_\Delta = 15.4$, it can be shown that 44,000 lb would be needed in order to obtain the dolos stability observed during the previous model tests.

Thus it is not surprising that the 22,000-lb dolosse sustained significant damage when exposed to the severe wave conditions of Hydrograph I. Since there was only minor damage to the harbor-side armor stone of Section 1 and some instability of the sea-side dolosse should be expected for the test conditions selected in this study, POD decided not to conduct further tests at sta 19+50.

Sta 14+00

19. The existing prototype breakwater cross section at sta 14+00 consists of 500- to 2,000-lb core material overlaid with one layer of keyed and fitted 16,000- to 20,000-lb armor stone on a 1V-on-1.5H sea-side slope and a 1V-on-1H harbor-side slope. The structure has a crown elevation of +11.0 ft mllw and a crown width of 21 to 22 ft. The sea-side slope was rehabilitated in 1977 by placing 1,000- to 4,000-lb stone on a 1V-on-2H slope from the toe to the crown. This stone was then overlaid with two layers of 22,000-lb dolosse. The dolos protection extends from the -16.0 ft mllw toe to the breakwater crown. Construction of the dolos armor layers used special toe placement and random placement on the remainder of the sea-side slope. Either during original construction or subsequent to that time, an apron of 13- to 350-lb stone has built up on a 1V-on-6H slope to an elevation of -4.3 ft mllw on the harbor side of the breakwater.

20. Section 2, Plate 5 and Photos 8-10 (before-test photographs are of the second test section; no before-test photographs were taken of the first and third test sections), was constructed in the test flume at a scale of 1:31 and reproduced, as closely as possible, the existing conditions at sta 14+00. The 22,000-lb dolosse were represented in the model as having individual prototype weights of 23,318 lb. The existing 16,000- to 20,000-lb armor stone were represented as having average individual weights of 18,000 lb with a linear gradation of ±15 percent by weight. The test setion was exposed to the wave and swl conditions of Hydrograph II. Minor to moderate in-place rocking of several dolosse on the lower sea-side slope was noted throughout the test but no displacement occurred. All three hydrograph steps produced significant overtopping, but the more severe overtopping conditions of Step 3 caused the displacement of one harbor-side slope armor stone. This displacement occurred early in Step 3 and no other damage occurred during the

remainder of the test. The overall condition of the breakwater appeared good at the end of the test (Photos 11-13).

21. The test section was rebuilt and once again exposed to the test conditions of Hydrograph II. For this test, the hydrograph steps were reversed to see what effect, if any, this would have on the test results. (This hydrograph step reversal had been done during testing of Section 1 and no effects were noted on the overall test results.) Step 3 produced extensive damage on the crown and harbor-side slope. Early in Step 3, two of the keyed and fitted armor stone were displaced (one from the crown and one from just below the swl). Following this displacement, the crown and harbor-side slope began to unravel with each successive wave. One dolos was displaced from the crown onto the harbor-side rubble stone. By the end of Step 3, damage to the crown and harbor-side slope had slowed down, but the majority of the crown stone had been displaced and the overtopping waves had displaced a significant amount of core material. The armor-stone crown had failed, but this did not result in any undermining or displacement of dolosse along the sea side of the crown. Although the crown and harbor-side slope were considered failed, the structure was exposed to Steps 2 and 1 to see if these conditions would displace any of the crown dolosse or cause any other additional damage. These conditions caused no additional damage, but some moderate in-place rocking of dolosse on the lower sea-side slope and a slight shift toward the harbor-side were noted on a few of the crown dolosse. By the end of the test, all damage had stopped. The sea-side dolosse were in good condition and the armor-stone crown and harbor-side slope were in need of total rehabilitation (Photos 14-16).

22. It was noted during the construction of the second test section that the keyed and fitted construction achieved was not as tight as had been reproduced on the first test section. This may very well have been the reason for the failure of the second test section. Not knowing exactly how tight the existing prototype keyed and fitted armor stone actually are, it was decided to rebuild the test section. During this building, an effort was made to achieve the same tightness of construction as had been accomplished on the first test section. The test section was once again exposed to Hydrograph II. The hydrograph steps were run in reverse to see if the tighter crown and harbor-side slope construction would prove stable for the selected test conditions. One armor stone was displaced from the upper harbor-side slope

during Step 3 and minor to moderate in-place rocking of 10 to 12 dolosse was noted throughout the hydrograph. No other movement or damage occurred and the structure was in good condition at the end of the test (Photos 17-19).

23. Test results for Section 2 showed that the stability of the crown and harbor-side slope, when exposed to the selected test conditions (Hydrograph II), is highly dependent upon how tightly keyed and fitted the existing prototype armor stone actually is. If a tight construction exists, it appears from the model tests that the existing section should be adequate. On the other hand, if tightly keyed and fitted crown and harbor-side slopes do not exist at sta 14+00, the model tests showed that the prototype structure could sustain extensive damage if it is exposed to the selected test conditions. The existing dolosse on the sea-side slope at sta 14+00 are an adequate design, but if the crown and harbor-side slope were to sustain severe damage, this could result in the eventual undermining and failure of crown dolosse. Based on these results, it was decided that testing of the proposed crown and harbor-side rehabilitation design for sta 14+00 was warranted.

24. The crown and harbor-side rehabilitation design for sta 14+00 was proposed by POD and would consist of casting in place a concrete rib cap on the crown and adding one layer of uniformly placed tribars on the harbor-side slope of the existing (Section 2) breakwater. The rib cap would extend to an elevation of +16.0 ft mllw. The individual ribs would be 3 ft wide, 22 ft long (average), and 5 ft high (average) and would be connected together on 6-ft centers by 2- by 2- by 3-ft concrete sections. The harbor-side slope would be flattened to 1V on 1.5H using randomly placed 1,000- to 3,000-lb stone fill overlaid by one layer of uniformly placed 13,000-lb tribars, extending from the crown to an elevation of -7.3 ft mllw. The tribar toe would be trenched into the existing 13- to 350-lb rubble such that no more than one-third of tribar height extends above the trench.

25. The proposed rehabilitation was added to the existing model test section of sta 14+00 (Section 2). The model construction was carried out to reproduce, as closely as possible, the conditions described in paragraph 24. Section 2 with the added rehabilitation work is referred to as Section 2A, Plate 6 and Photos 20-22. For purposes of the model tests, it was agreed by WES and POD that the prototype, cast in-place rib cap could be assumed to be stable; therefore it was not necessary that the model rib cap be dynamically similar to its prototype counterpart. The model rib cap, constructed out of

Plexiglas, was geometrically similar to the proposed prototype rib cap and was held in place in the model, thus ensuring proper transmission, reflection, and dissipation of wave energy and the assumed stability of the prototype rib cap. Due to model scale selection and availability of model tribar unit weights, the 13,000-lb tribars were represented in the model as having individual weights of 12,827 lb. The test section was exposed to wave and swl conditions of Hydrograph II. Minor to moderate degrees of rocking in place of several dolosse on the lower sea-side slope were observed throughout the test, but no dolos displacement occurred. The rib cap reduced the amount of overtopping wave energy, but all three hydrograph steps still produced moderate to significant amounts of overtopping. The majority of the overtopping wave energy was deflected away from the harbor-side slope by the rib cap. Some very minor in-place rocking of a couple of tribars next to the rib cap was noted during Step 3. No other movement was observed and the structure was in very good condition at the end of the test (Photos 23-25).

Sta 10+00

26. The existing prototype breakwater cross section at sta 10+00 consists of 500- to 2,000-lb core material overlaid with one layer of keyed and fitted 16,000- to 20,000-lb armor stone on a 1V-on-1.5H sea-side slope and a 1V-on-1H harbor-side slope. The original construction has a crown height and width of +10.0 ft mllw and 22 ft, respectively. In 1977, the sea-side slope was rehabilitated with 14,000- to 24,000-lb armor stone. The armor stone was randomly placed on a 1V-on-2H slope from the -8.0 ft mllw toe to an elevation of +12.0 ft mllw. The cover-layer thickness tapers from two layers at the toe to one layer on the upper slope. Either during original construction or subsequent to that time, an apron of 13- to 350-lb stone has built up on a 1V-on-6H slope to an elevation of approximately -3.3 ft mllw on the harbor side of the breakwater.

27. Section 3, Plate 7 and Photos 26-28, was constructed at a scale of 1:25 and reproduced, as closely as possible, the existing conditions at sta 10+00. The 16,000- to 20,000-lb keyed and fitted armor stone and the 14,000- to 24,000-lb random-placed armor stone were represented as having average individual stone weights of 18,000 lb and 19,000 lb, respectively, with linear gradations of ±15 percent by weight. Section 3 sustained no

damage during its exposure to Hydrograph III (Photos 29-31). The only movement observed was minor in-place rocking of four to six armor stone on the lower sea-side slope. Steps 1 and 2 of Hydrograph III produced minor to moderate amounts of splash over the structure crown, while Step 3 produced very minor amounts of solid water overtopping.

PART IV: DISCUSSION

28. Based on linear wave theory, a shoaling coefficient can be calculated to predict the change in wave height that occurs as a wave propagates from deep water to some specified shallow-water depth (U. S. Army CERC 1977). This shoaling coefficient is dependent upon the wave period (T) and water depth (d) and is independent of the bottom slope over which it is propagating. This theory holds true for small amplitude waves, but is not valid for large wave heights and even less accurate for waves that are near breaking. Experimental data (U. S. Army CERC 1977) have shown that the relative wave height (H/d), where H is the wave height of other than small amplitude waves, is dependent upon the bottom slope as well as the wave period and the water depth. In general, an increase in water depth or a decrease in either wave period or bottom slope, while the other two variables are held constant, will result in a decrease in H/d . These conditions were observed during calibration of the test facility for the study reported herein. For Hydrograph I, Plate 1 and Table 1, H/d values of 1.0, 1.03, and 1.07 were measured at a 23-ft water depth for 12-, 14-, and 16-sec waves, respectively, which propagated over a 1V-on-10H bottom slope. For the same wave periods and bottom slope, measurements of H/d values made at a water depth of 12 ft were 1.13, 1.23, and 1.27. For Hydrograph III, Plate 3 and Table 3, H/d values of 0.89, 0.93, and 0.97 were measured at a 12-ft water depth for 12-, 14-, and 16-sec waves, respectively, which propagated over a 1V-on-24H bottom slope.

29. Subsequent to calibration and initiation of testing on the existing breakwater section at sta 19+50, Section 1, hydrographic surveys revealed that some of the bathymetry seaward of the breakwater, between sta 19+00 and 20+00, was flatter than the 1V-on-10H slope being used for the model tests. Based on the considerations presented and the measured data presented in paragraph 28, for those areas of the breakwater that are forwarded by a slope flatter than 1V on 10H, a smaller H/d value and thus a smaller wave height than that used during testing of Section 1 can be expected to occur at a prototype depth of 23 ft. This smaller wave height would most likely cause less damage to the existing dolosse than was observed during these model tests, but this decrease cannot be quantified based on the model tests conducted and reported herein. Step 3 of Hydrograph I consisted of a 16-sec, 24.5-ft breaking wave. This corresponds to a stability coefficient of 23.4 for the 22,000-lb dolosse.

Hypothetically, if it is assumed that a flatter slope in front of the breakwater would result in a 10 percent decrease in the H/d value for the 16-sec waves in 23 ft of water, this would correspond to a wave height of 22.3 ft. This in turn would result in a stability coefficient of 17.5 or less damage to the structure.

30. Based on observations of the tests of the proposed crown and harbor-side rehabilitation design at sta 14+00, Section 2A, it was evident that the concrete rib cap provided major protection for the harbor-side slope.

PART V: CONCLUSIONS

31. Based on the test and results reported herein, it is concluded that for:

a. Sta 19+50.

(1) The existing 22,000-lb sea-side dolosse on Section 1 are not an adequate design for the wave and swl conditions of Hydrograph I. Based on items discussed in paragraph 29, this may or may not be a significant prototype problem.

(2) The existing 35,600-lb tribars on the upper sea-side, the crown, and the 16,000- to 20,000-lb armor stone on the harbor-side slope of Section 1 are adequate designs for the wave and swl conditions of Hydrograph I.

(3) All three steps of Hydrograph I produce significant wave overtopping on Section 1.

b. Sta 14+00.

(1) If the existing 16,000- to 20,000-lb armor stone on the crown and harbor-side slope of Section 2 have a tight keyed and fitted construction, it appears that the prototype is an adequate design for the wave and swl condition of Hydrograph II.

(2) If the existing crown and harbor-side slope do not have a tight keyed and fitted construction, it is possible that the structure could sustain extensive damage if exposed to the conditions of Hydrograph II.

(3) The existing 22,000-lb sea-side dolosse armor on Section 2 is an adequate design for the condition of Hydrograph II, but if the crown and harbor-side slope sustain severe damage, this damage could result in the eventual undermining and displacement of dolosse along the sea side of the crown.

(4) The proposed concrete rib cap for the crown and 13,000-lb tribars for the harbor-side slope of Section 2A are very adequate designs for the wave and swl conditions of Hydrograph II.

(5) All three steps of Hydrograph II caused significant wave overtopping on Section 2. The proposed concrete rib cap for Section 2A reduced the wave overtopping produced by Hydrograph II, but the amount of overtopping still ranged from moderate to significant.

c. Sta 10+00.

(1) The existing 14,000- to 24,000-lb sea-side armor stone and 16,000-to 20,000-lb armor stone on the crown and harbor-side slope of Section 3 are adequate designs for the wave and swl conditions of Hydrograph III.

(2) Step 3 of Hydrograph III produced very minor amounts of wave overtopping on Section 3, while Steps 1 and 2 produced moderate amounts of splashover but no solid water overtopping.

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Table 1

Hydrograph I, Sta 19+50

Step	swl ft mllw	Test Wave Period sec	Test Wave Height* ft	Prototype Duration hr	Wave Type
	+4.0	12.0	12.0	0.25	Shakedown
1	+4.0	12.0	23.0	1.0	Worst breaking
2	+4.0	14.0	23.8	1.0	Worst breaking
3	+4.0	16.0	24.5	1.0	Worst breaking

* Measured at the -19.0 ft mllw contour (top of 1V-on-10H slope).

Table 2

Hydrograph II, Sta 14+00

Step	swl ft mllw	Test Wave Period sec	Test Wave Height* ft	Prototype Duration hr	Wave Type
	+4.0	12.0	10.25	0.25	Shakedown
1	+4.0	12.0	20.50	1.00	Worst breaking
2	+4.0	14.0	21.00	1.00	Worst breaking
3	+4.0	16.0	22.50	1.00	Worst breaking

* Measured at the -16.0 ft mllw contour (top of 1V-on-10H slope).

Table 3

Hydrograph III, Sta 10+00

Step	swl ft mllw	Test Wave Period sec	Test Wave Height* ft	Prototype Duration hr	Wave Type
	+4.0	12.0	6.0	0.25	Shakedown
1	+4.0	12.0	10.7	1.0	Worst breaking
2	+4.0	14.0	11.1	1.0	Worst breaking
3	+4.0	16.0	11.6	1.0	Worst breaking

* Measured at the -8.0 ft mllw contour (top of 1V-on-24H slope).

Photo 1. Comparison of random and special placement of toe dolosse

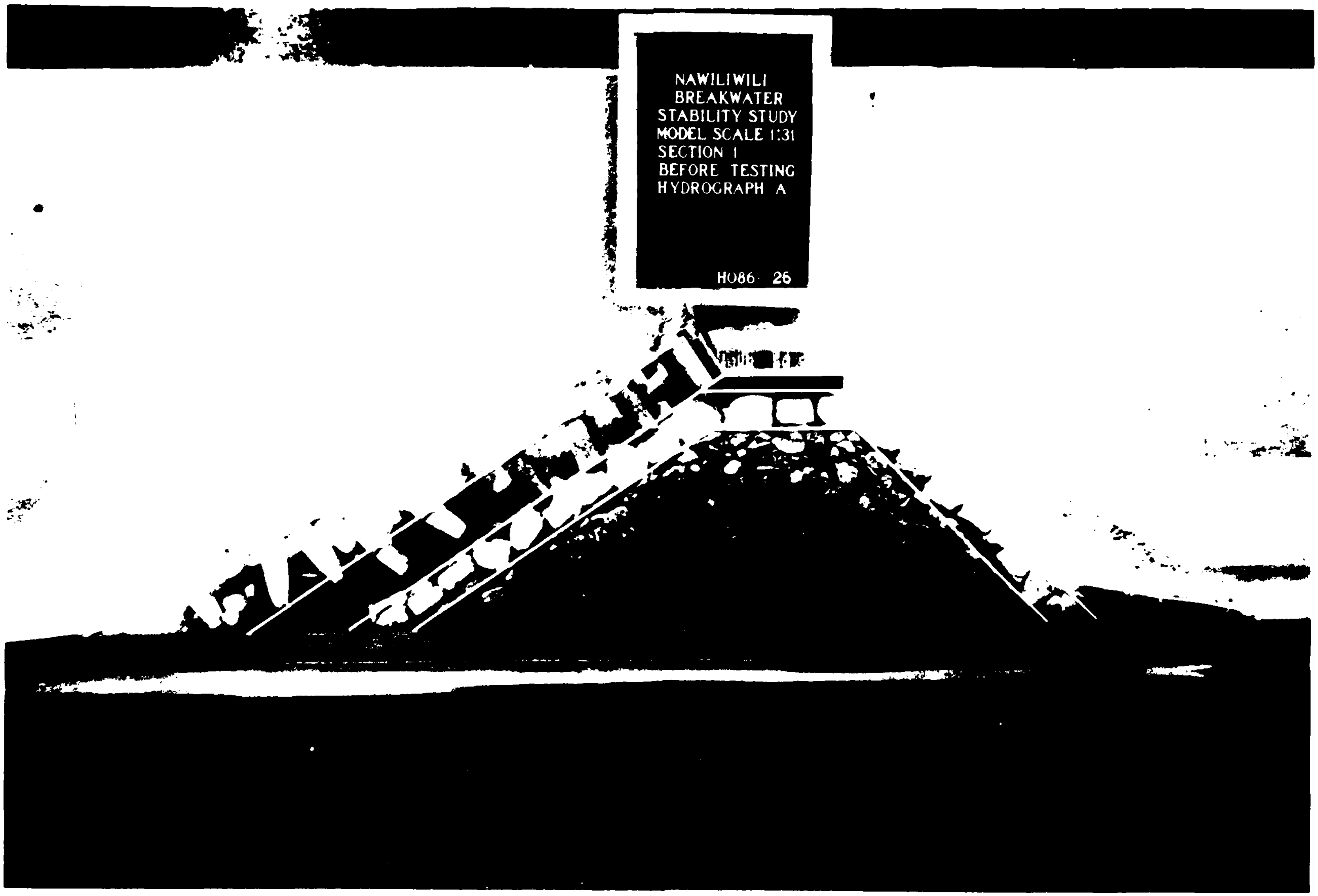


Photo 2. Side view of Section 1 before testing

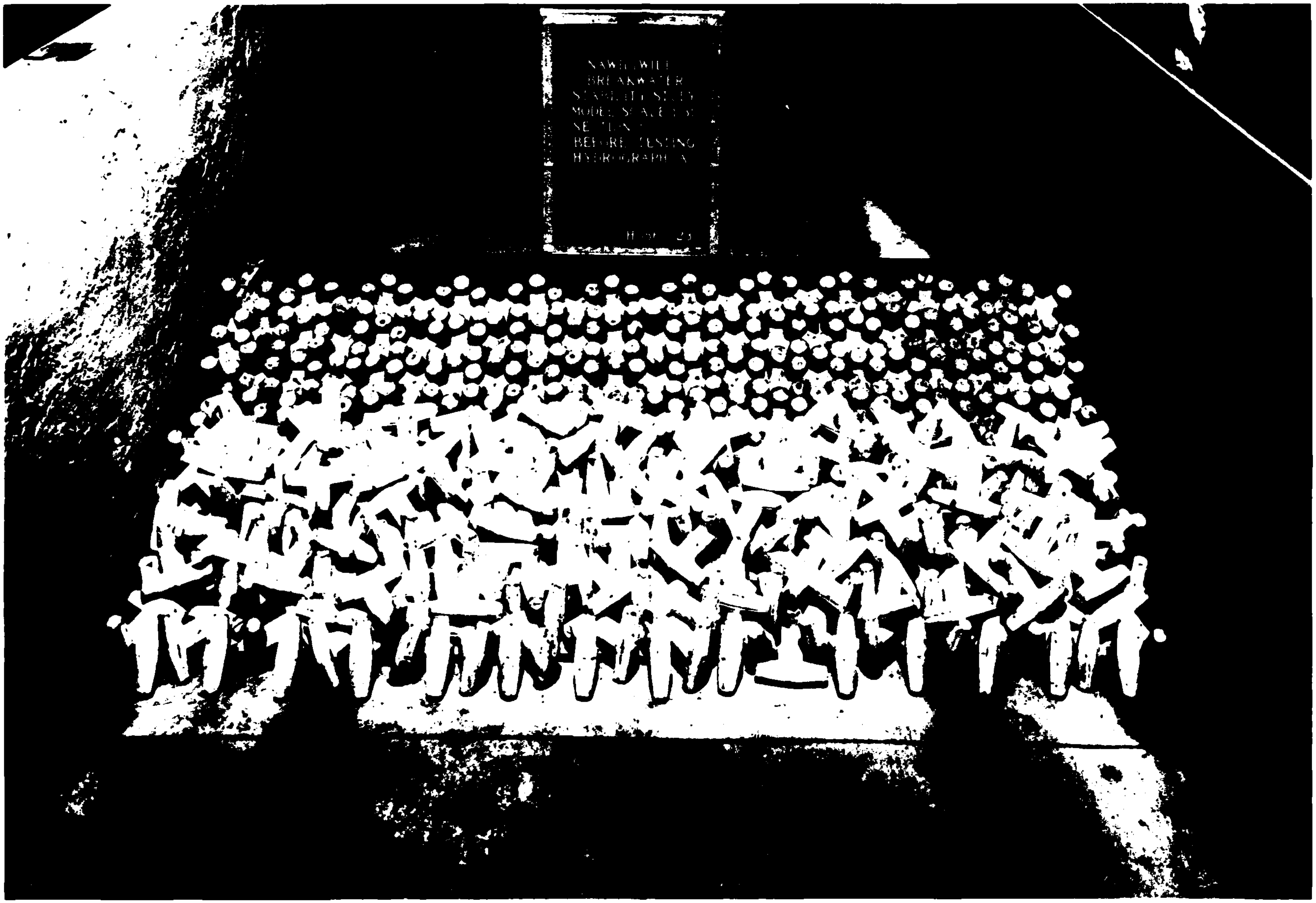


Photo 3. Sea-side view of Section 1 before testing

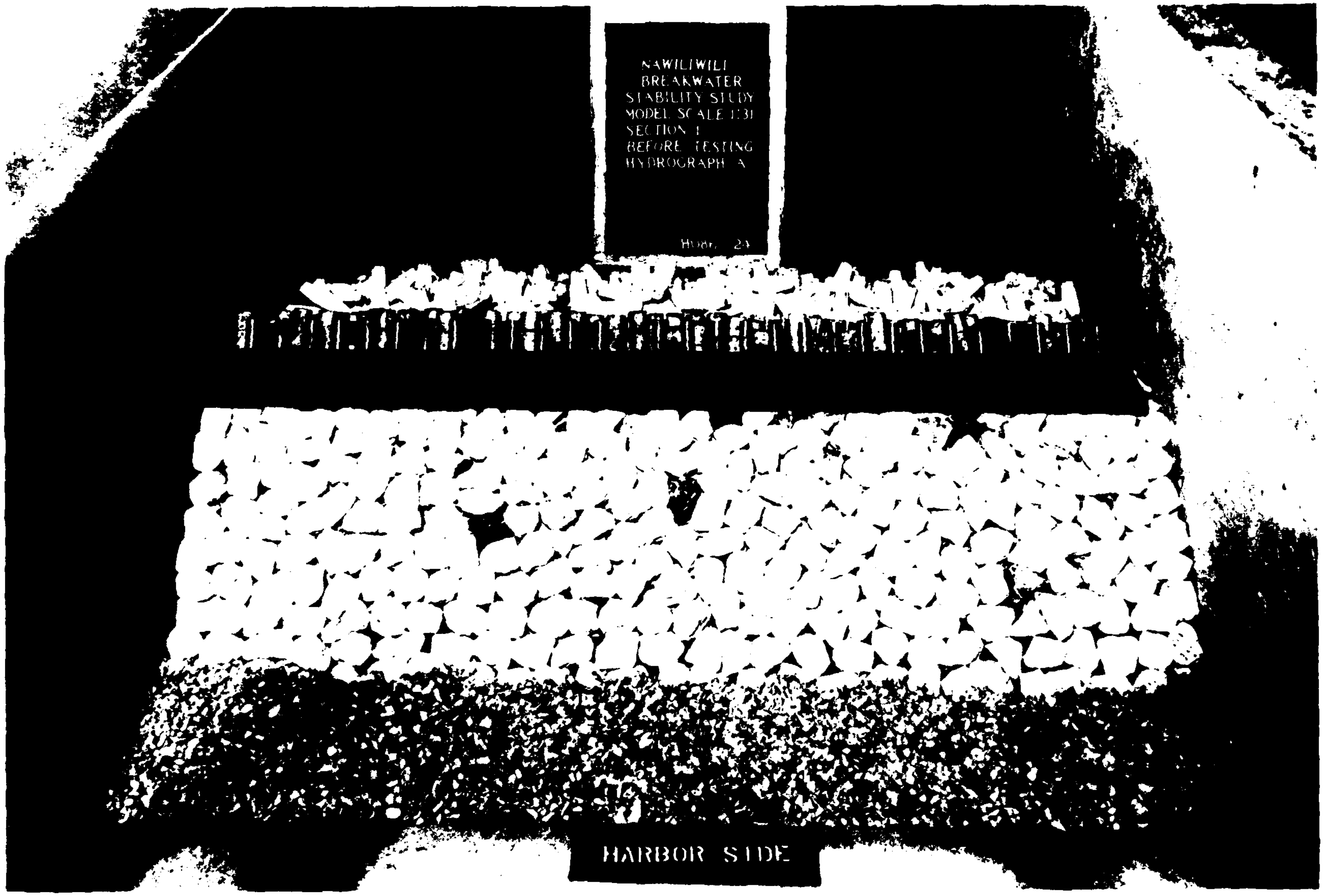


Photo 4. Harbor-side view of Section 1 before testing

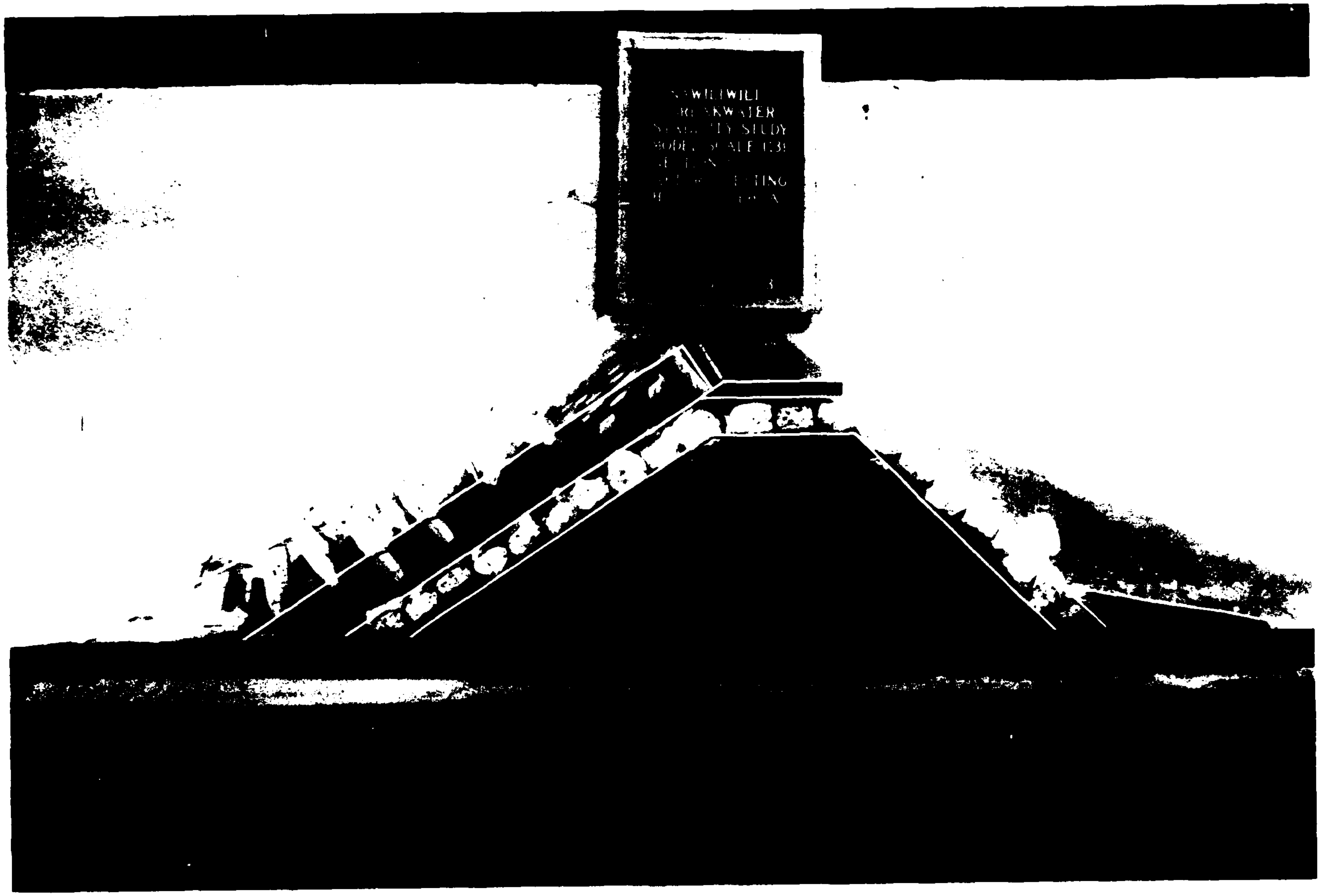


Photo 5. Side view of Section I after testing

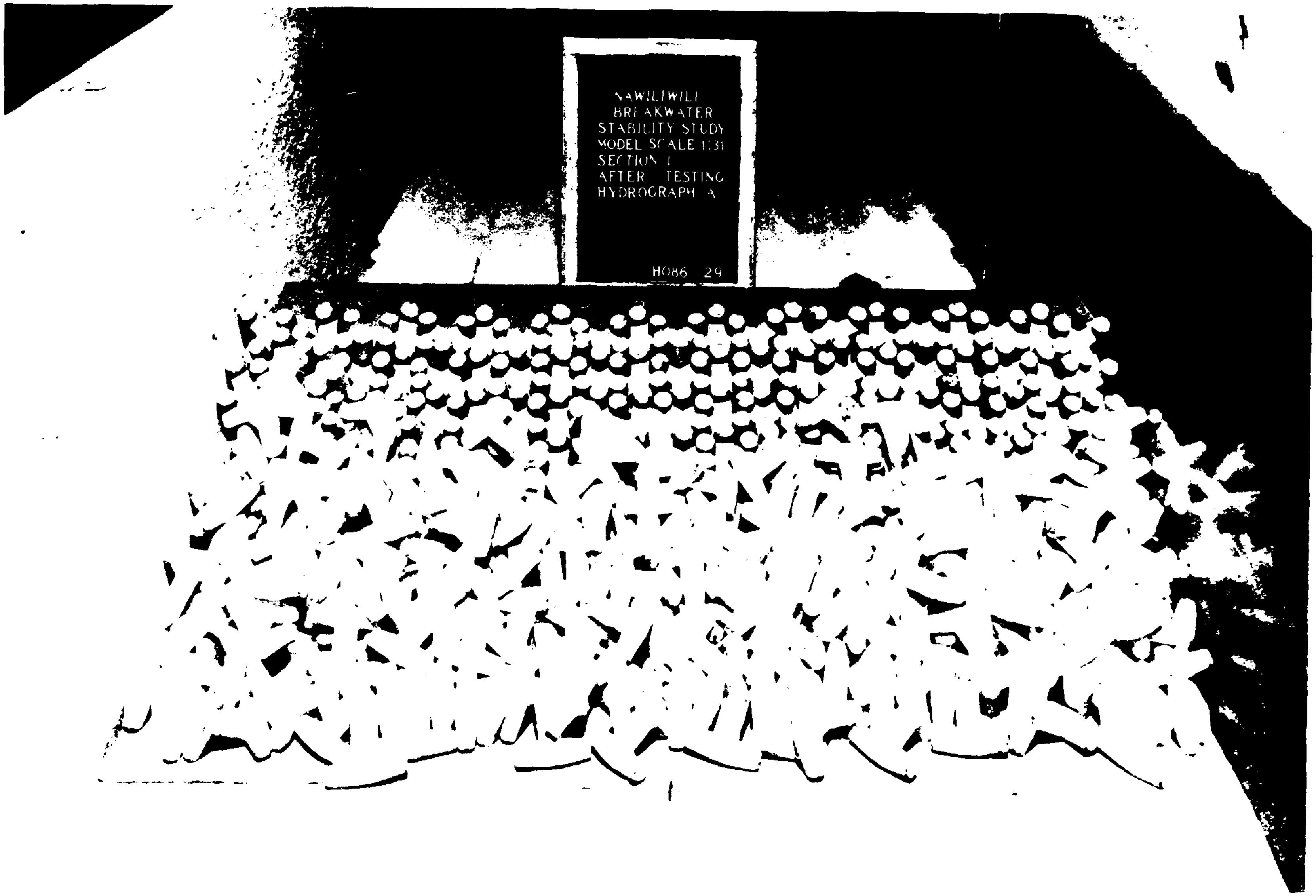


Photo 6. Sea-side view of Section 1 after testing

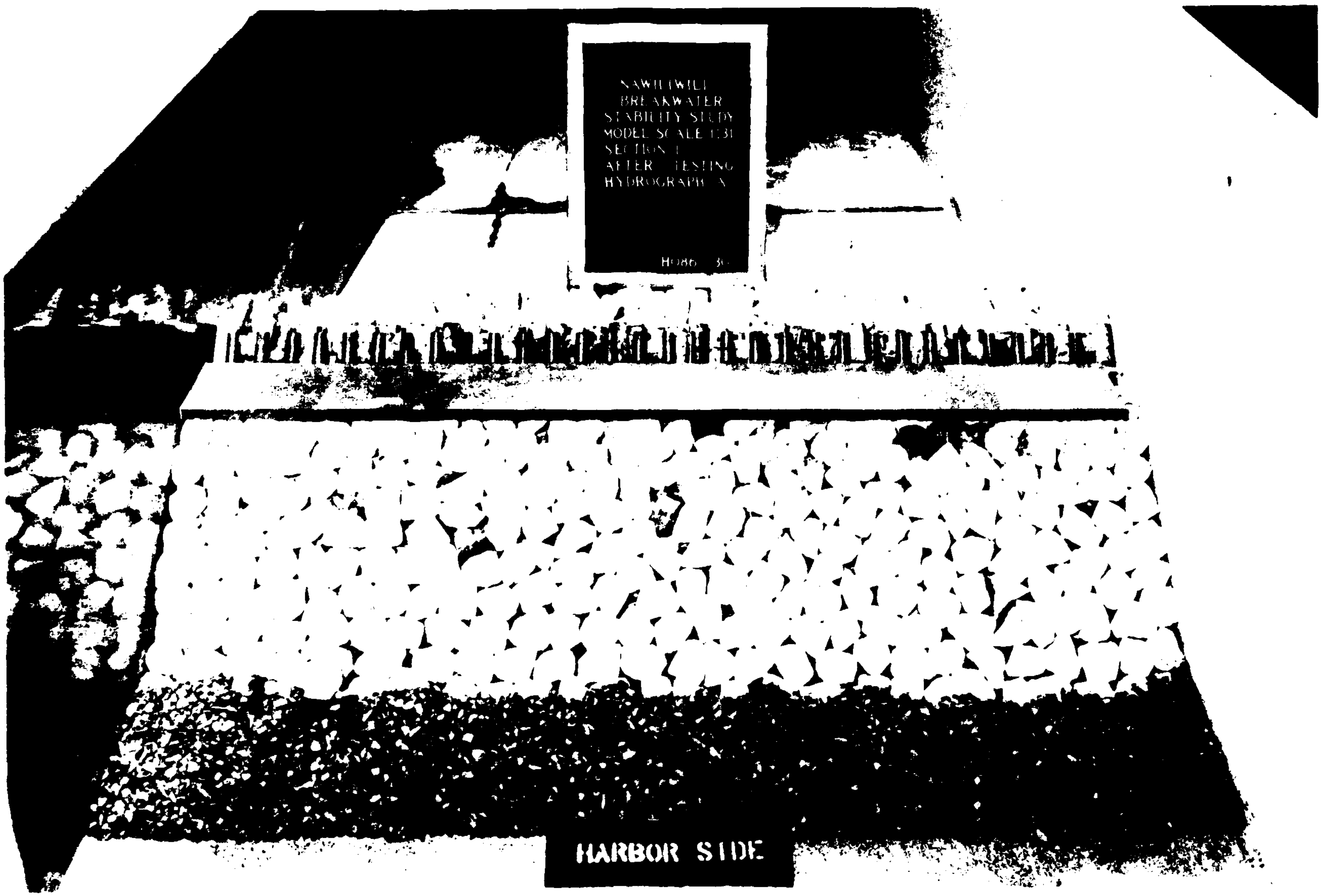


Photo 7. Harbor-side view of Section 1 after testing

Photo 8. Side view of Section 2 before testing, second test section

Photo 9. Sea-side view of Section 2 before testing, second test section

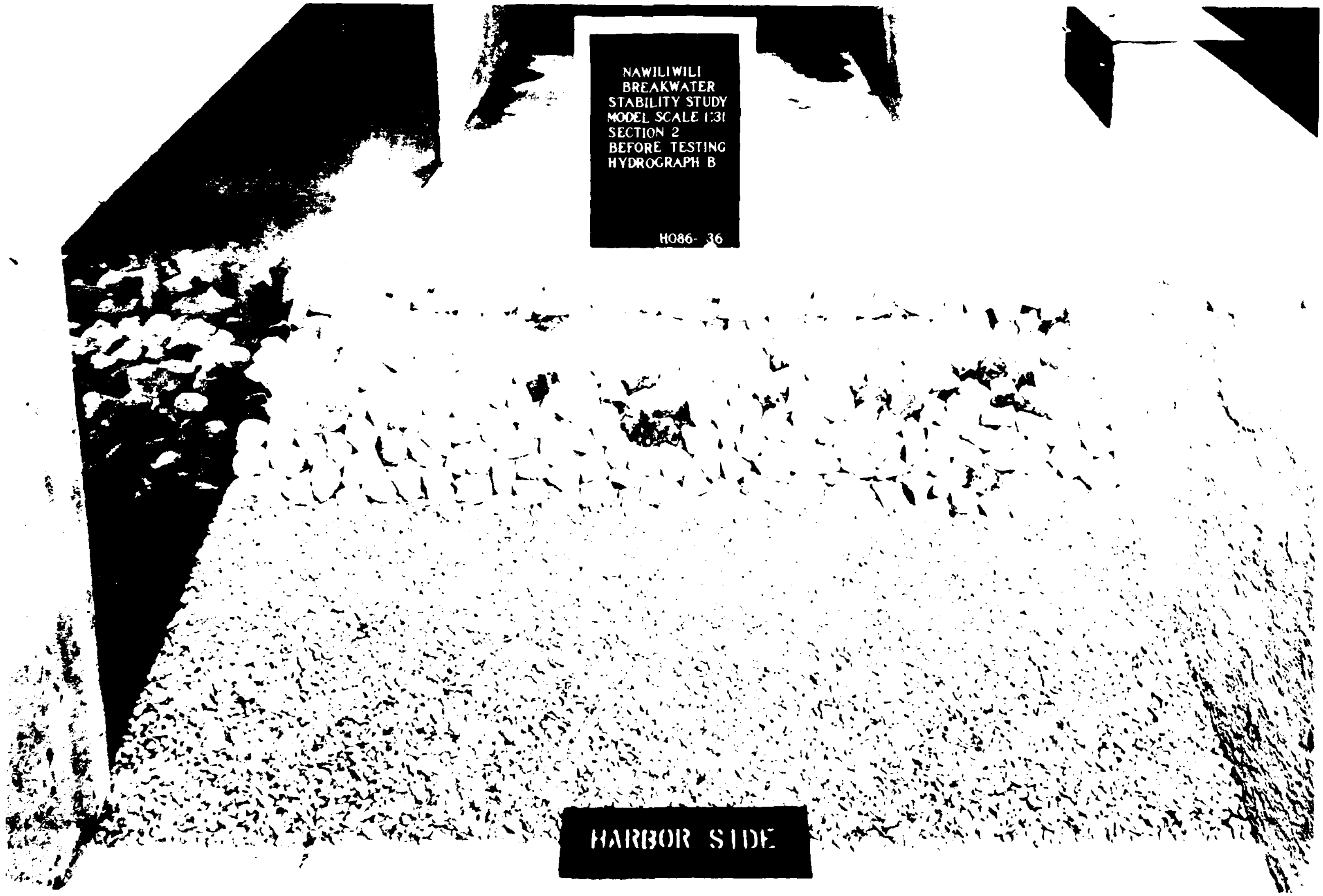


Photo 10. Harbor-side view of Section 2 before testing, second test section

Photo 11. Side view of Section 2 after testing, first test section

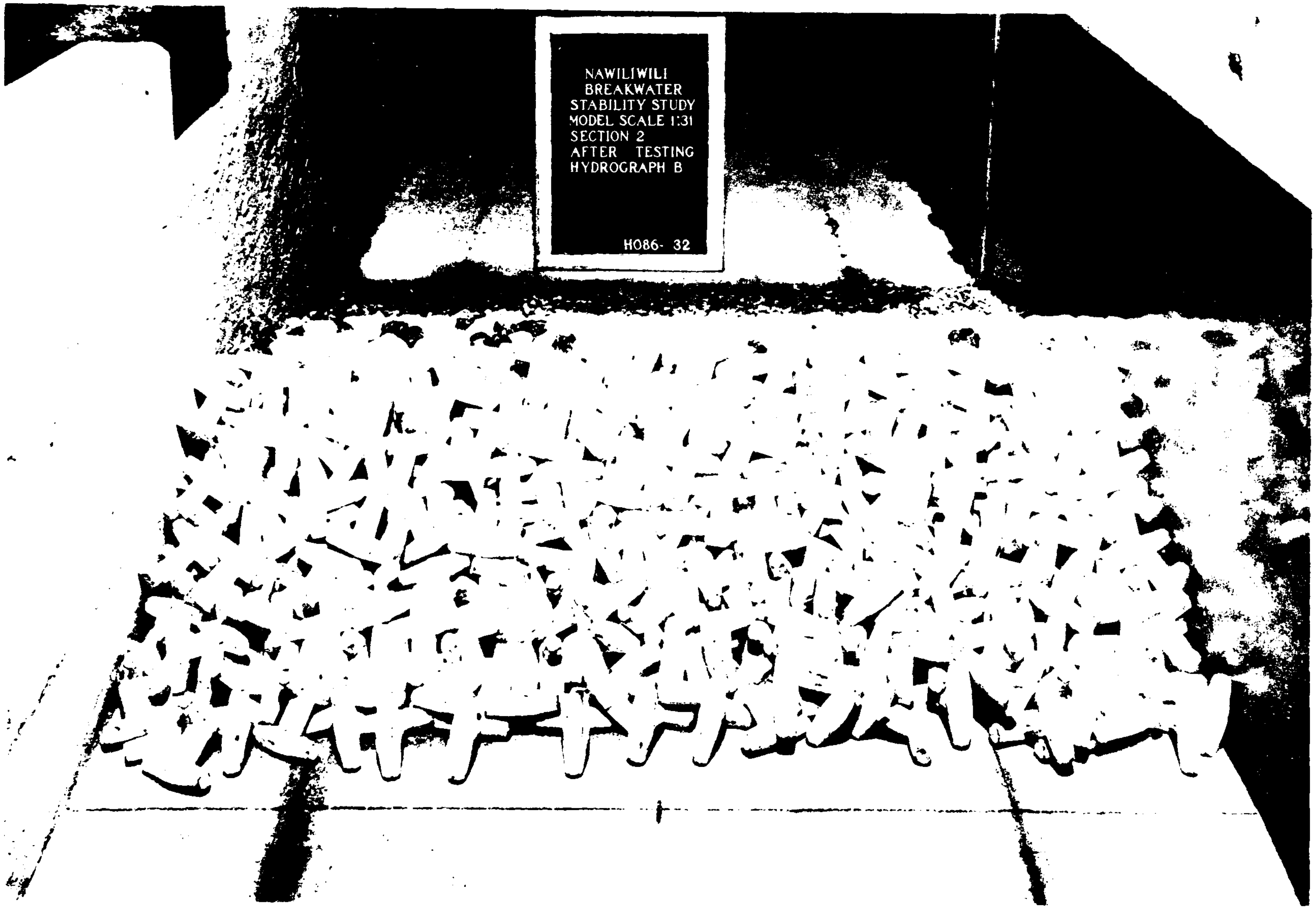


Photo 12. Sea-side view of Section 2 after testing, first test section

Photo 13. Harbor-side view of Section 2 after testing, first test section

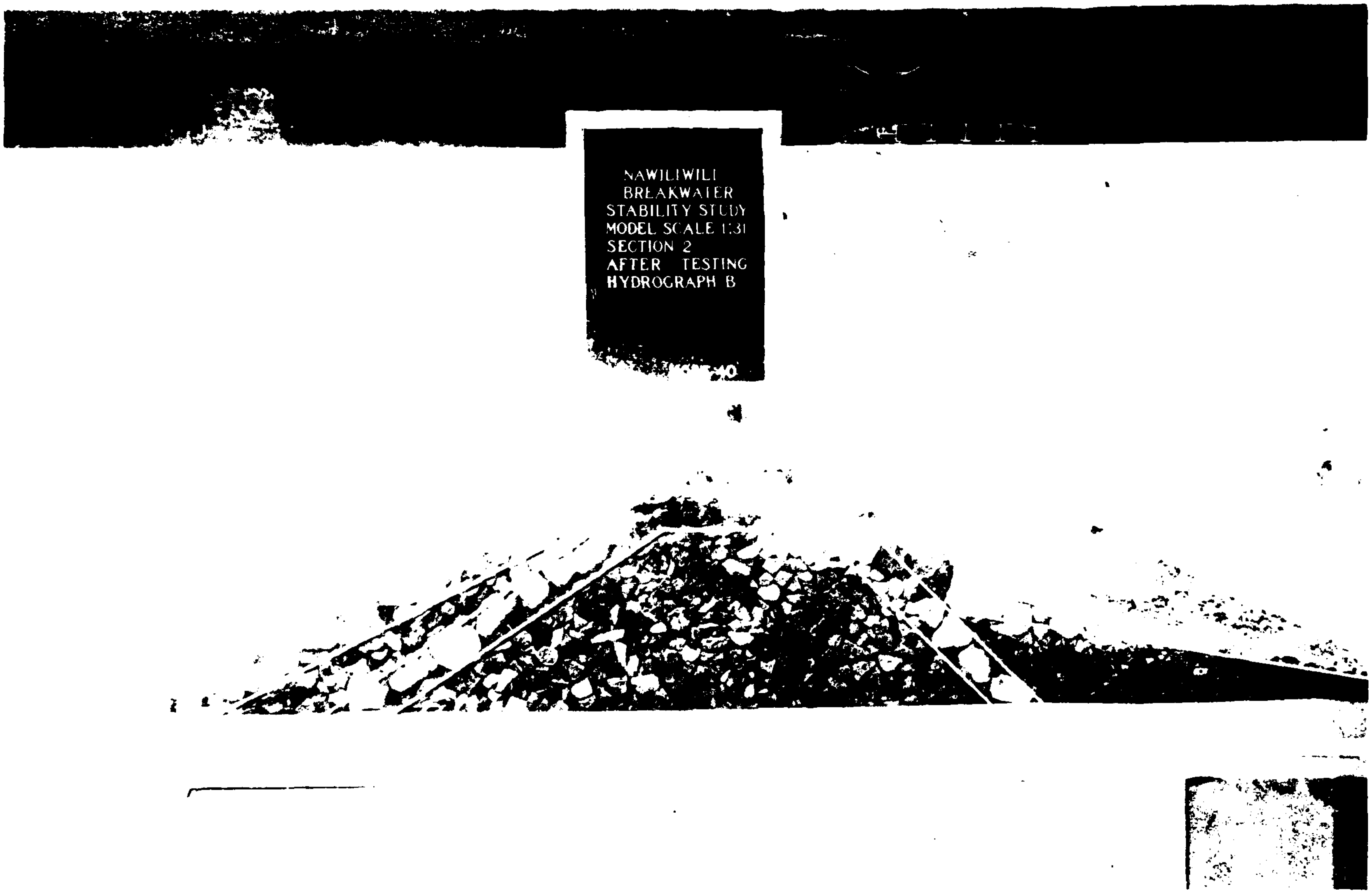


Photo 14. Side view of Section 2 after testing, second test section

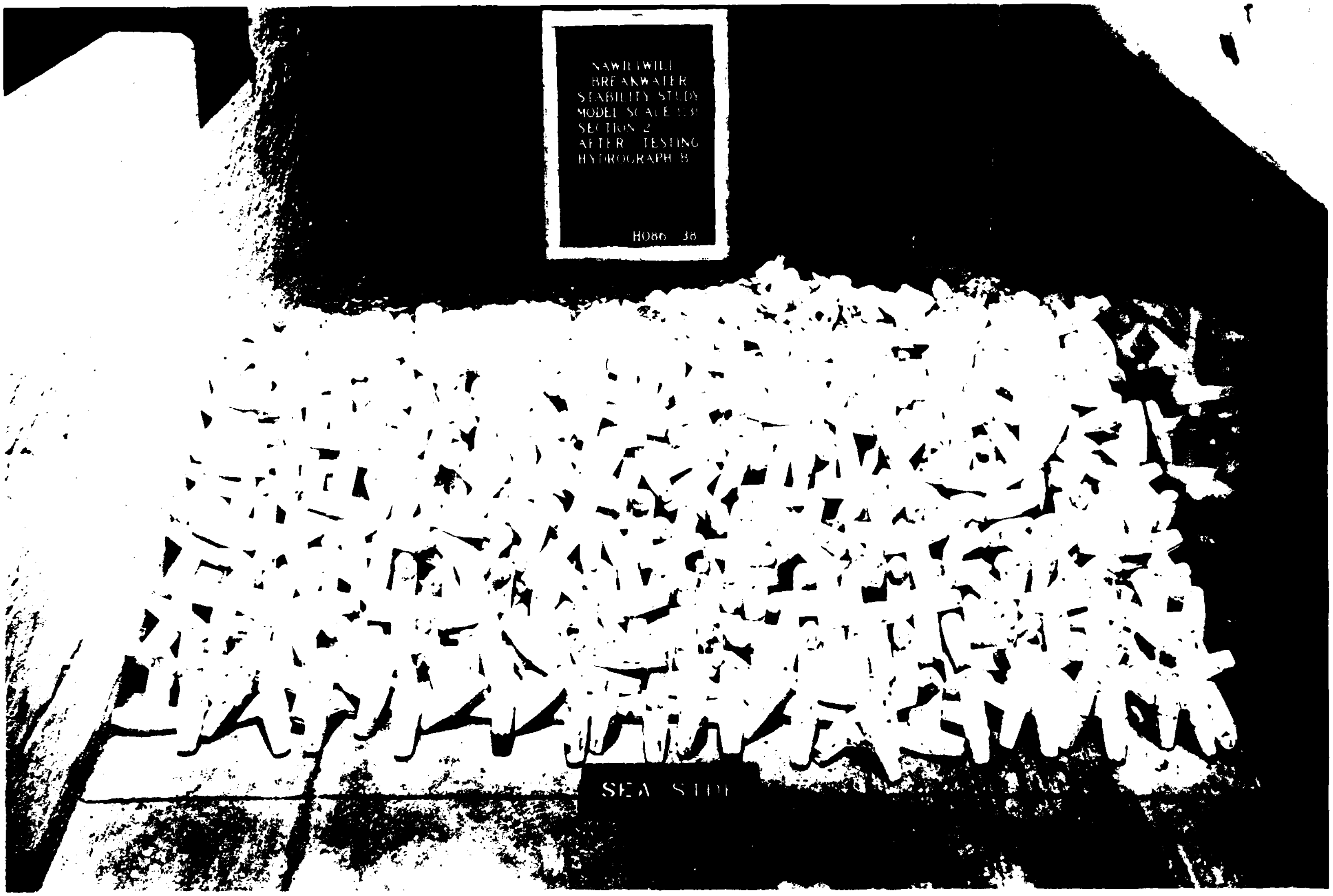


Photo 15. Sea-side view of Section 2 after testing, second test section

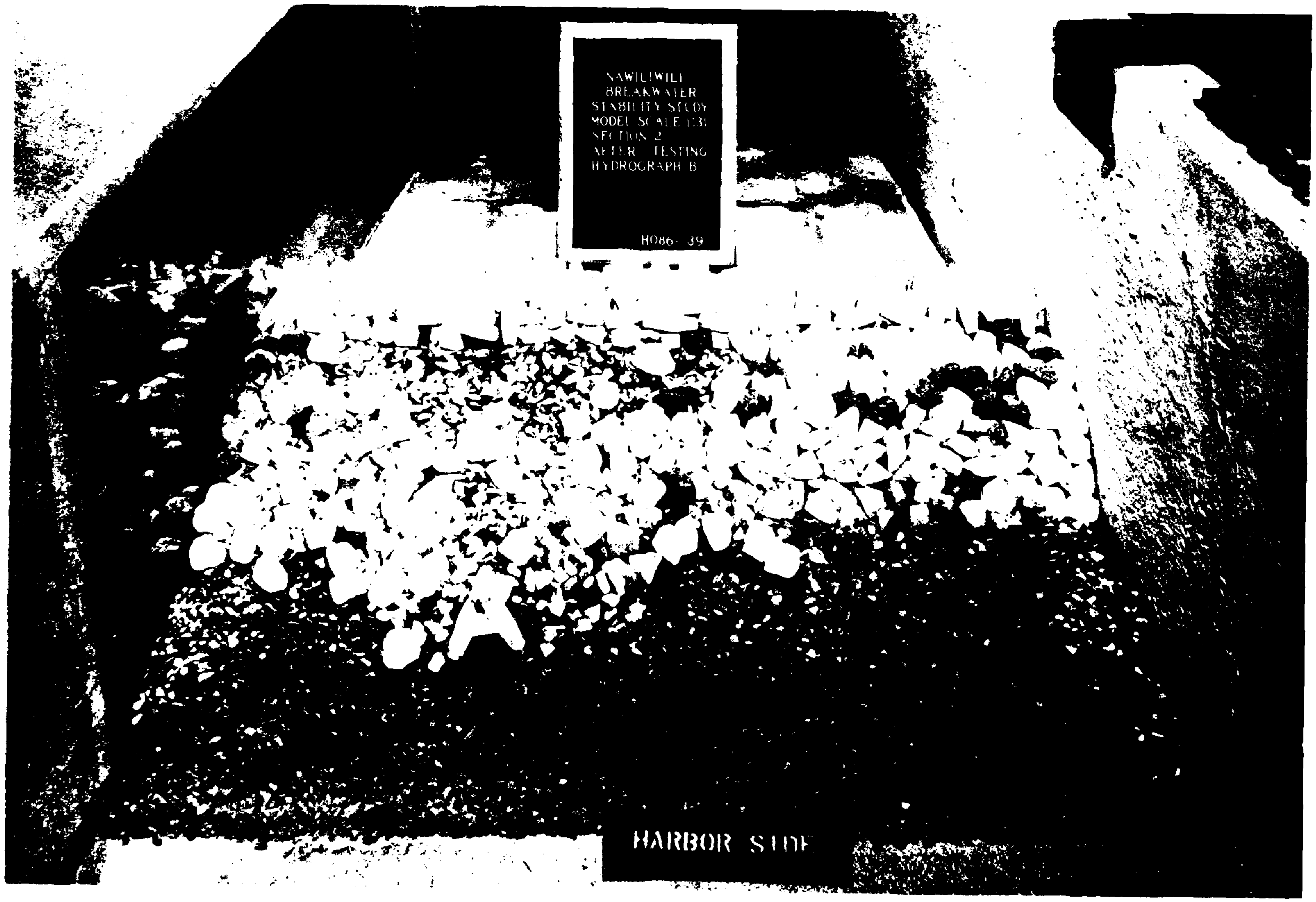


Photo 16. Harbor-side view of Section 2 after testing, second test section

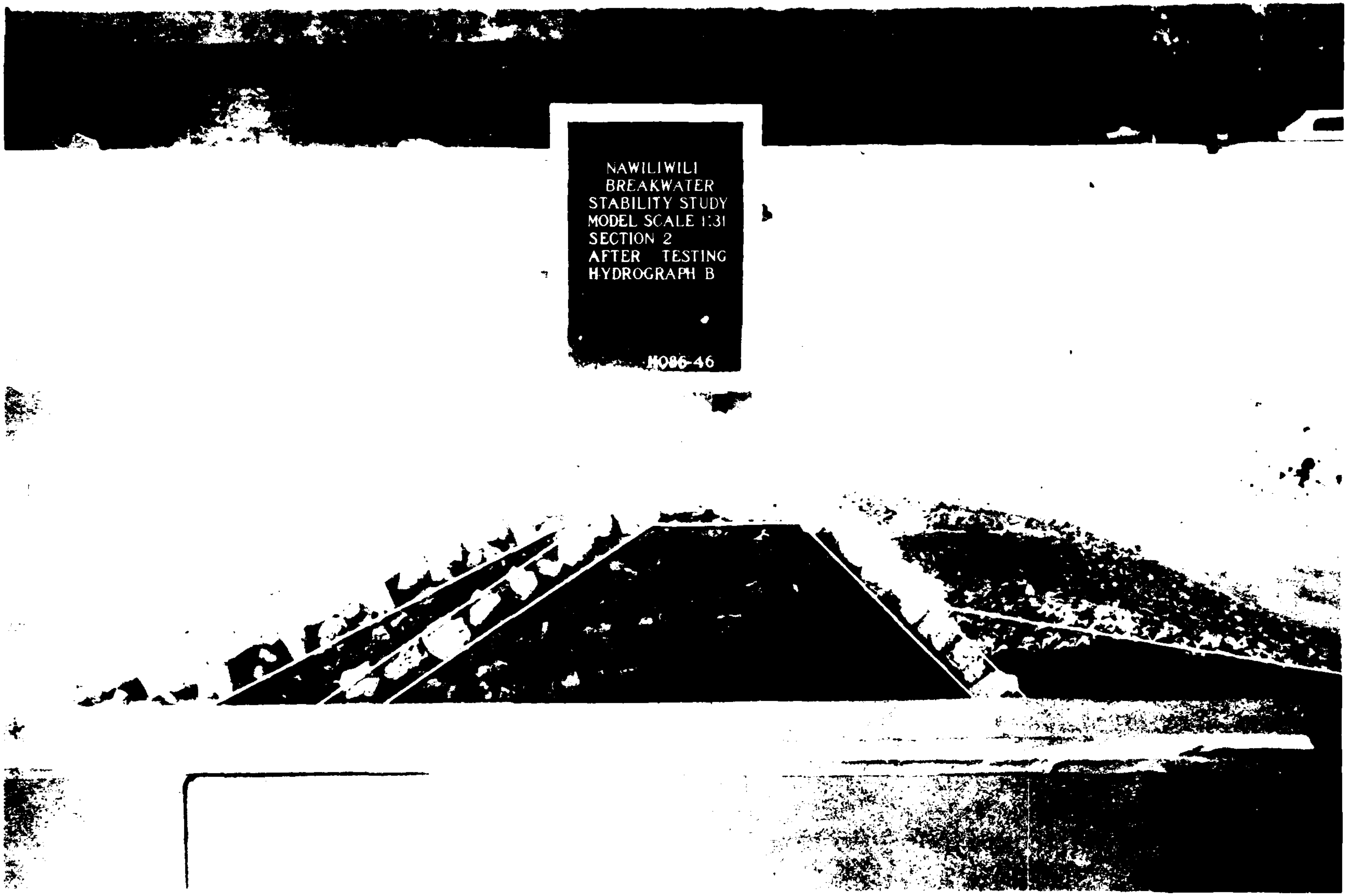


Photo 17. Side view of Section 2 after testing, third test section

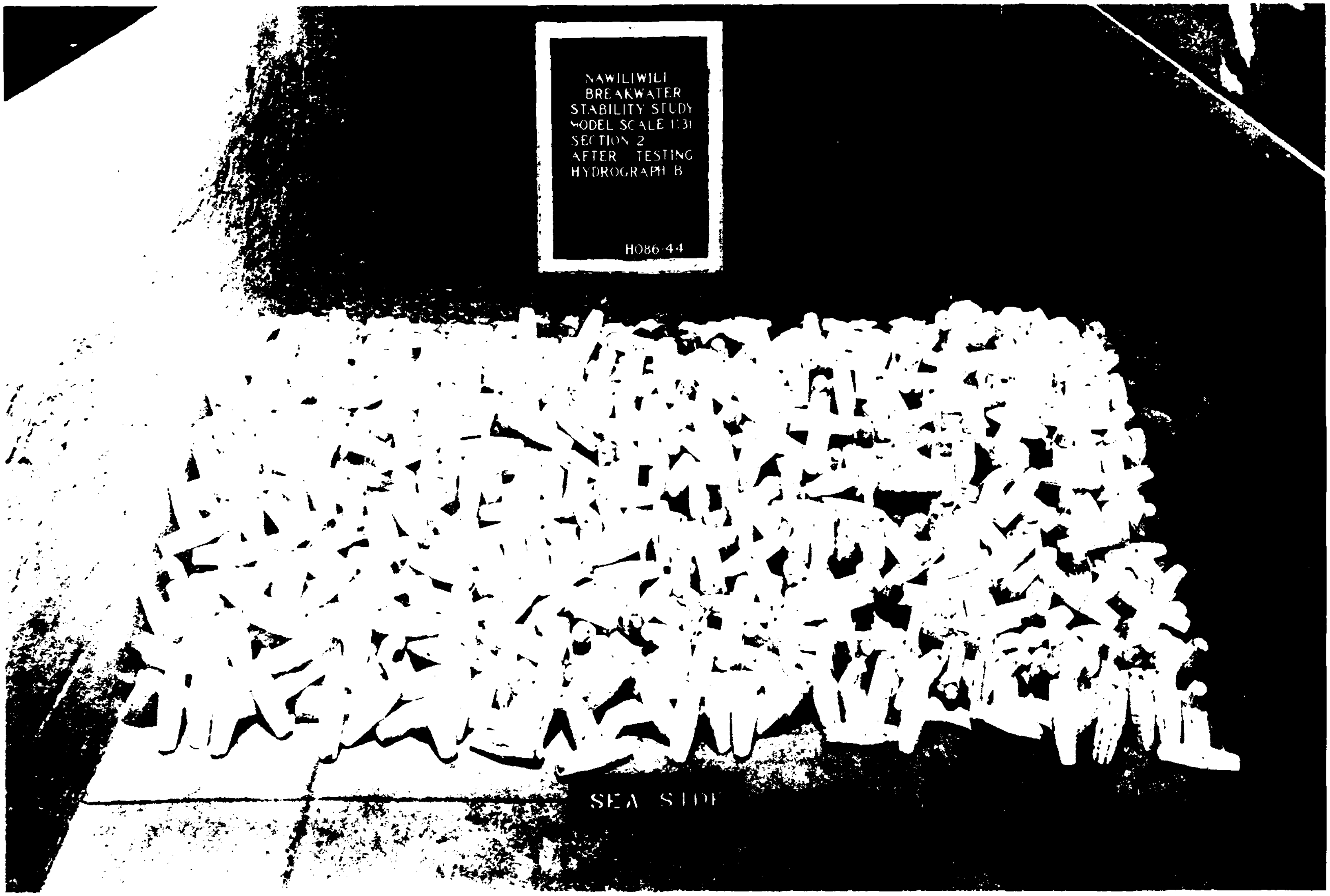


Photo 18. Sea-side view of Section 2 after testing, third test section

Photo 19. Harbor-side view of Section 2 after testing, third test section

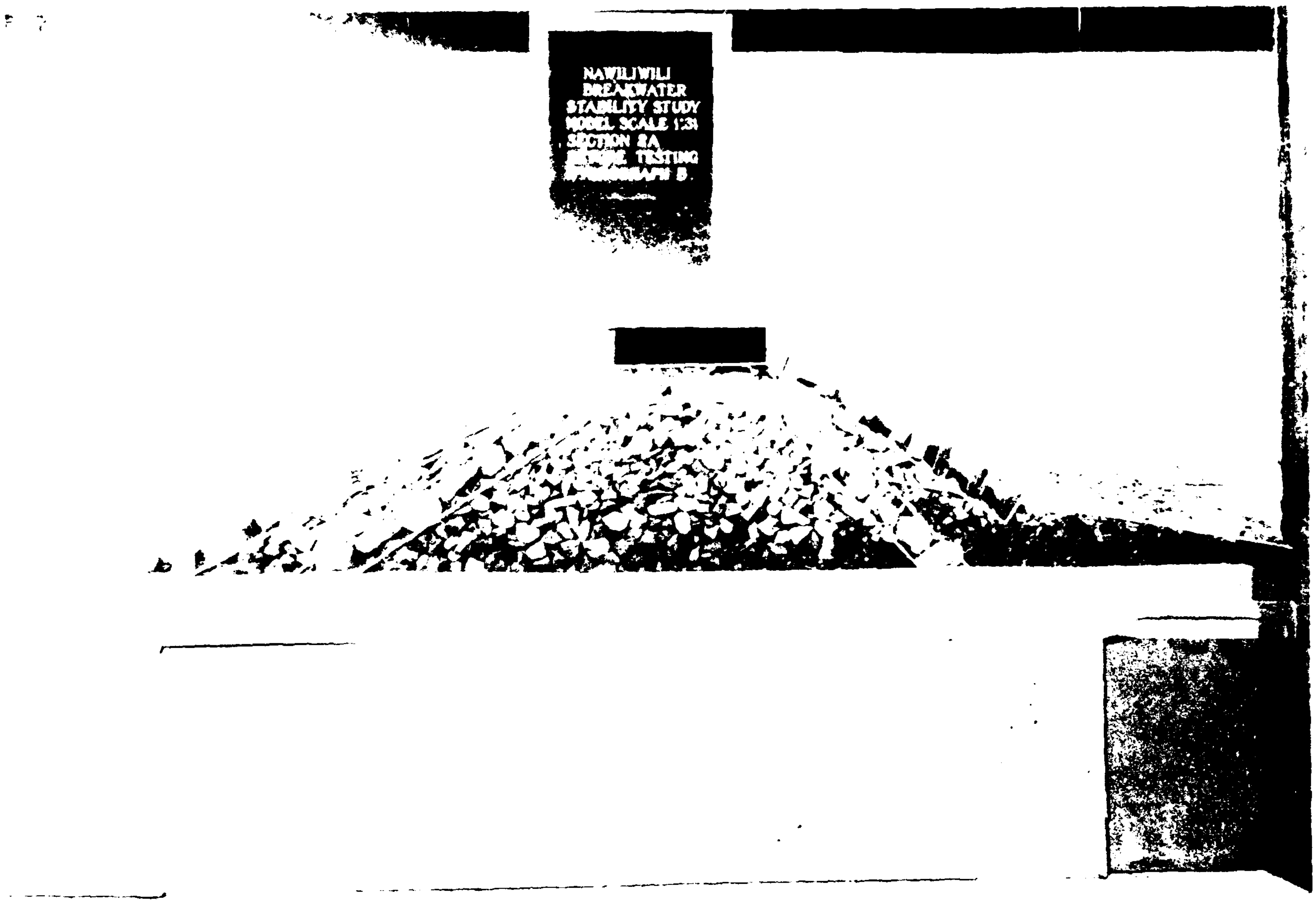


Photo 20. Side view of Section 2A before testing

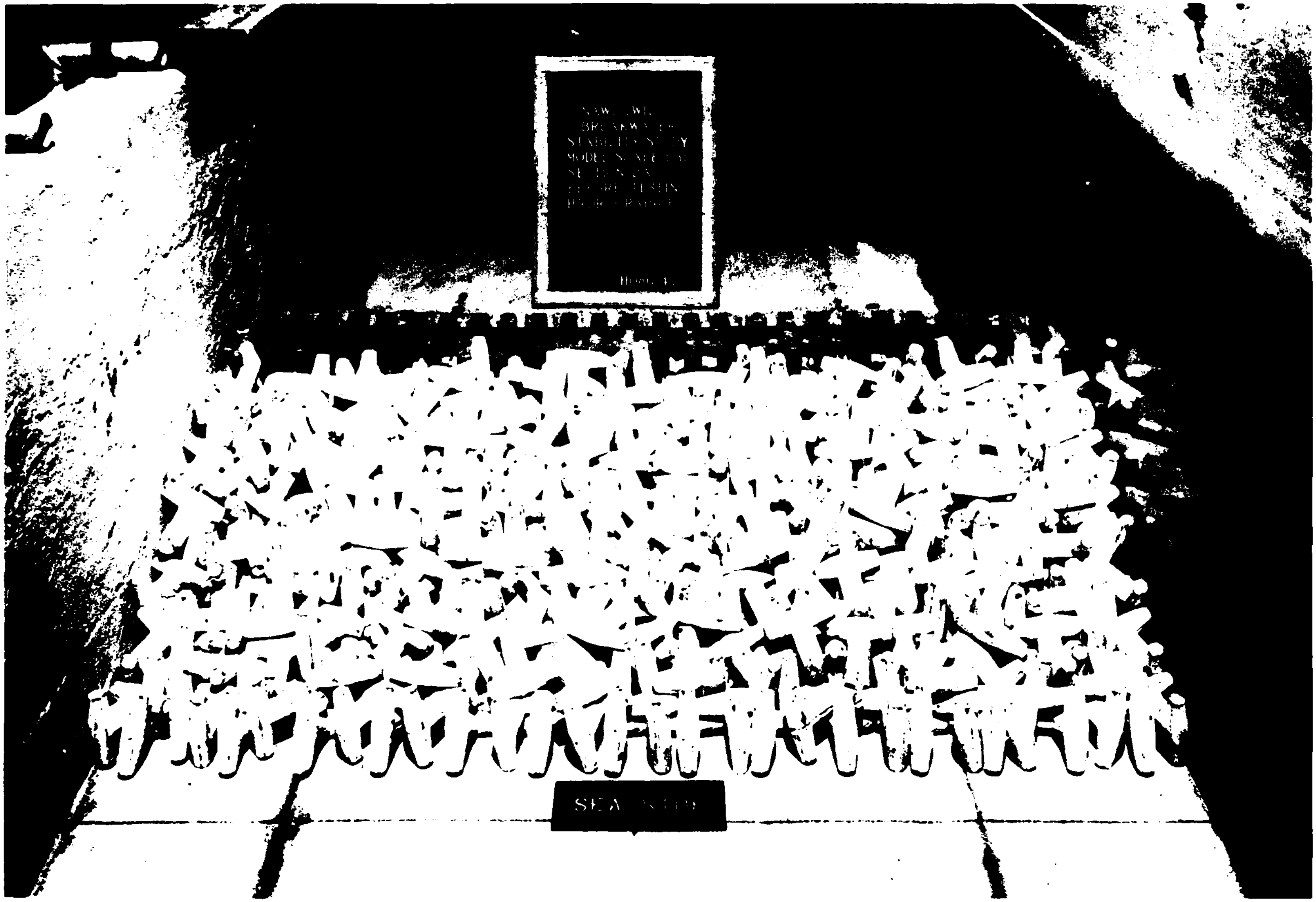


Photo 21. Sea-side view of Section 2A before testing

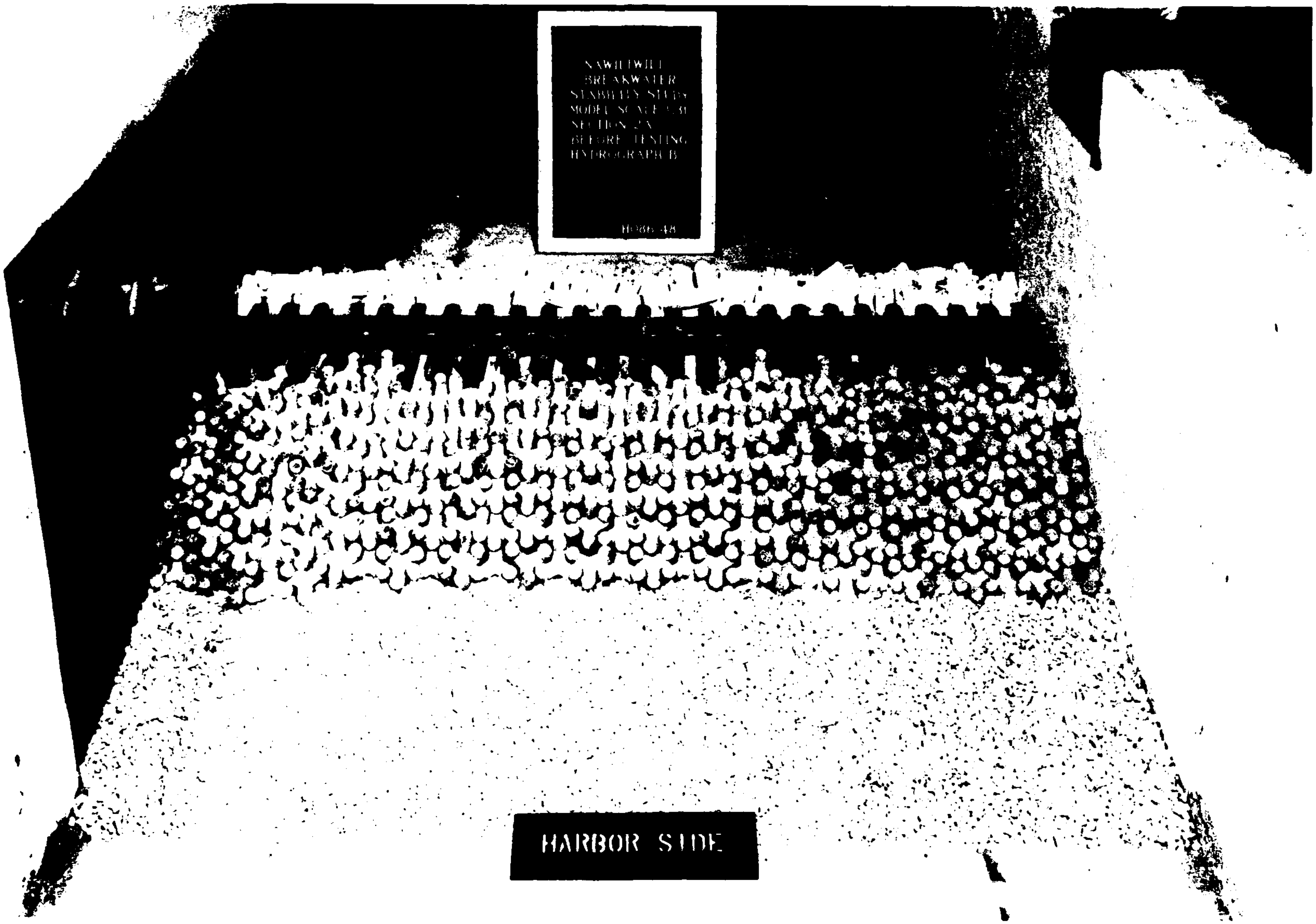


Photo 22. Harbor-side view of Section 2A before testing

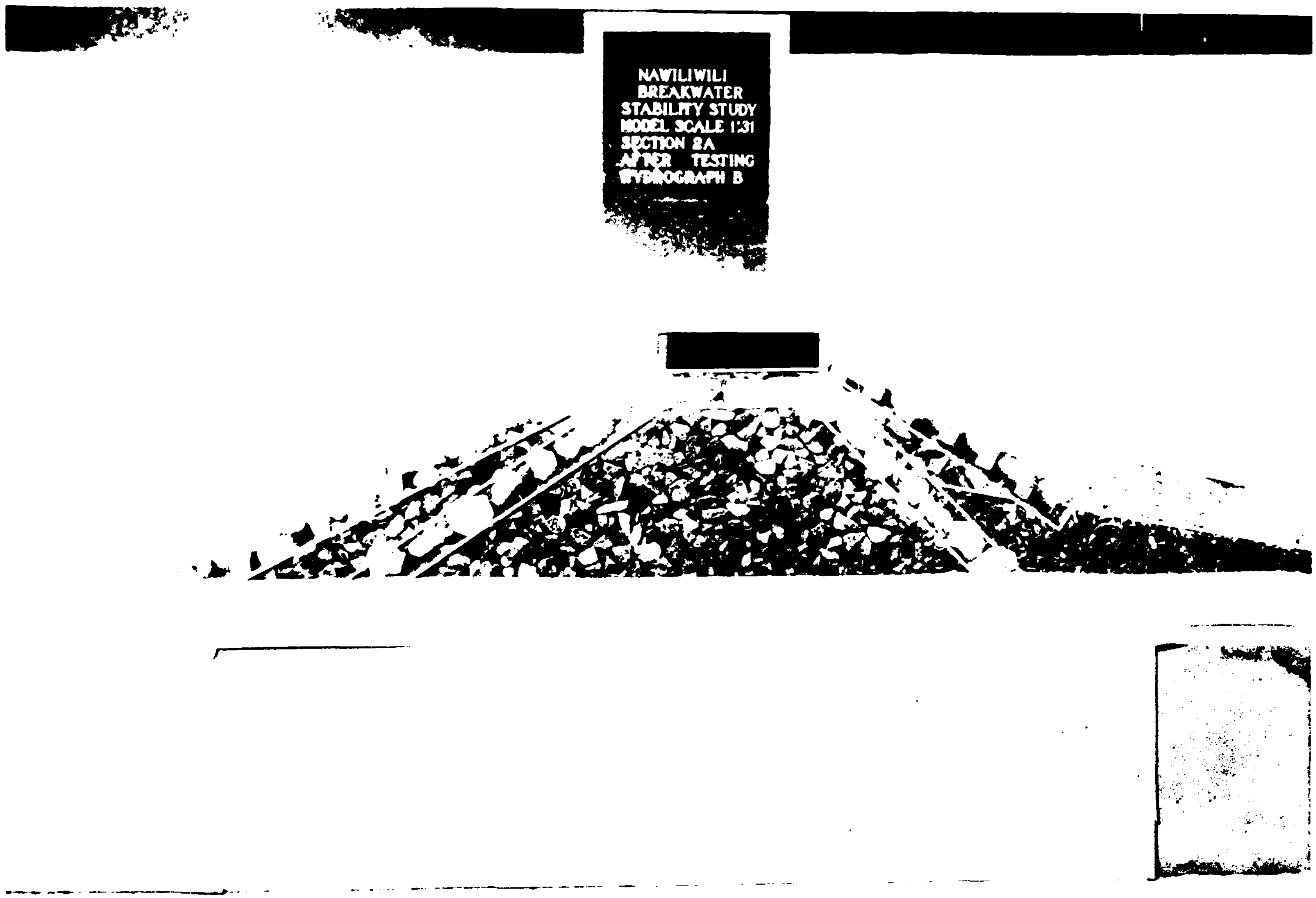


Photo 23. Side view of Section 2A after testing

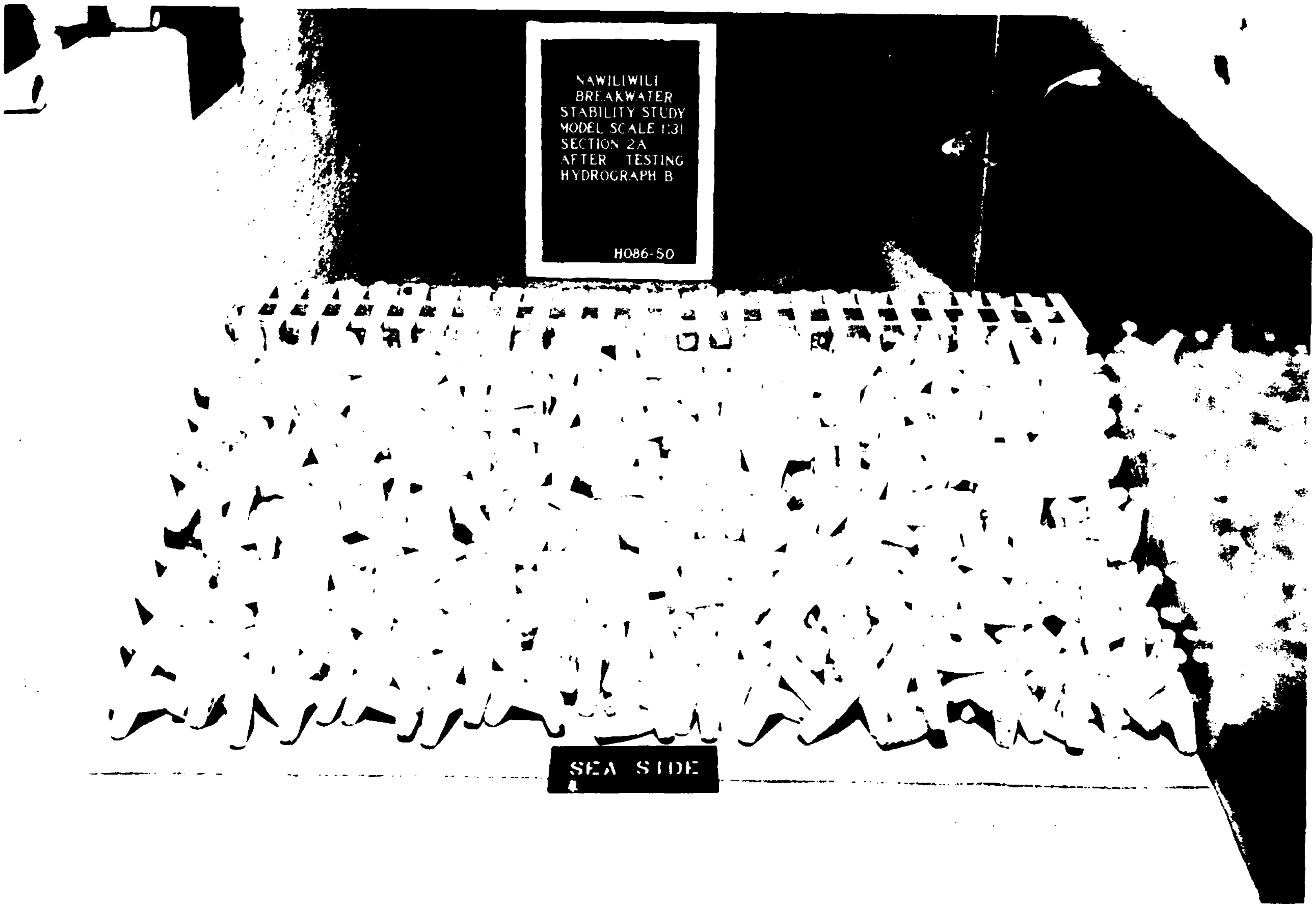


Photo 24. Sea-side view of Section 2A after testing

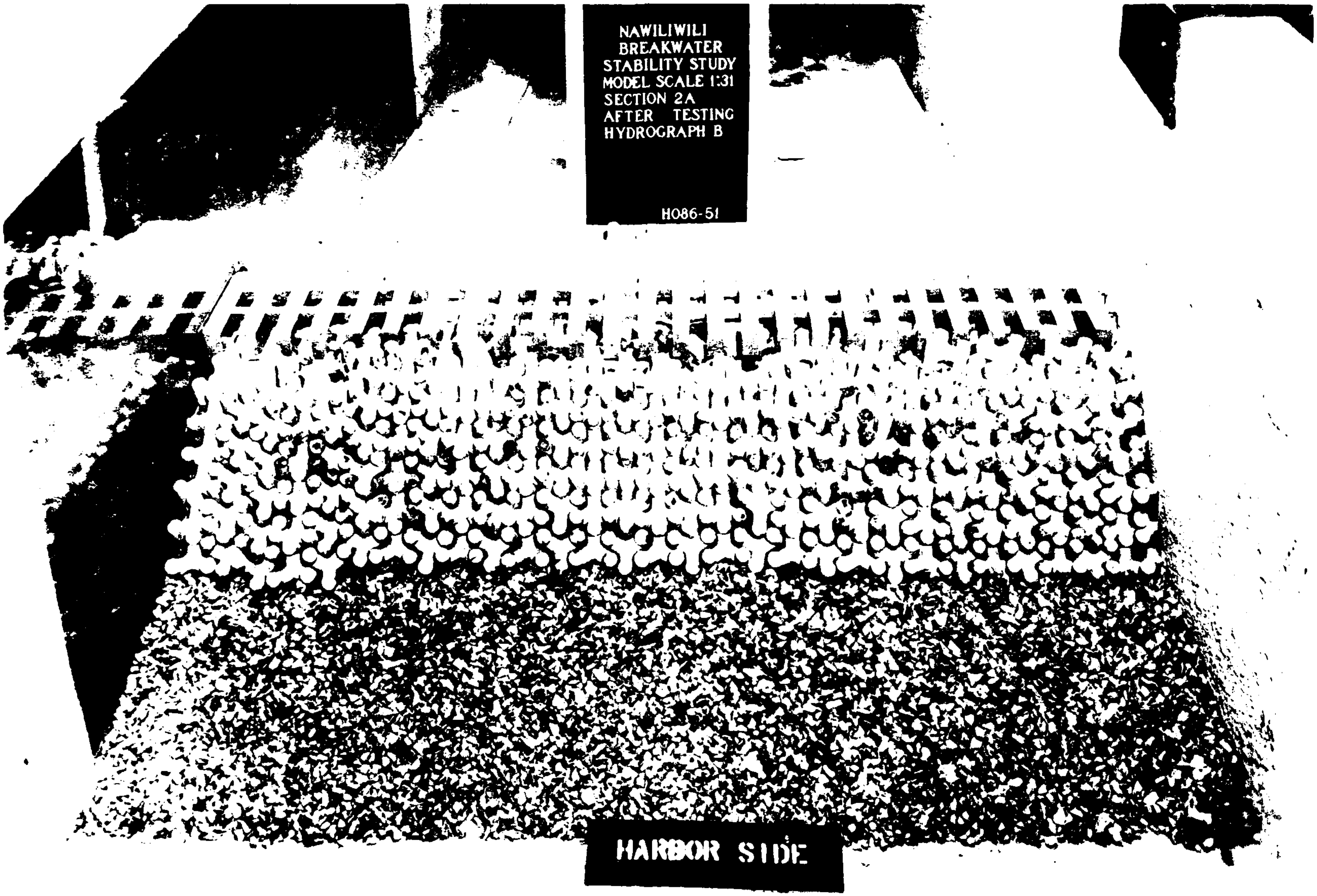


Photo 25. Harbor-side view of Section 2A after testing

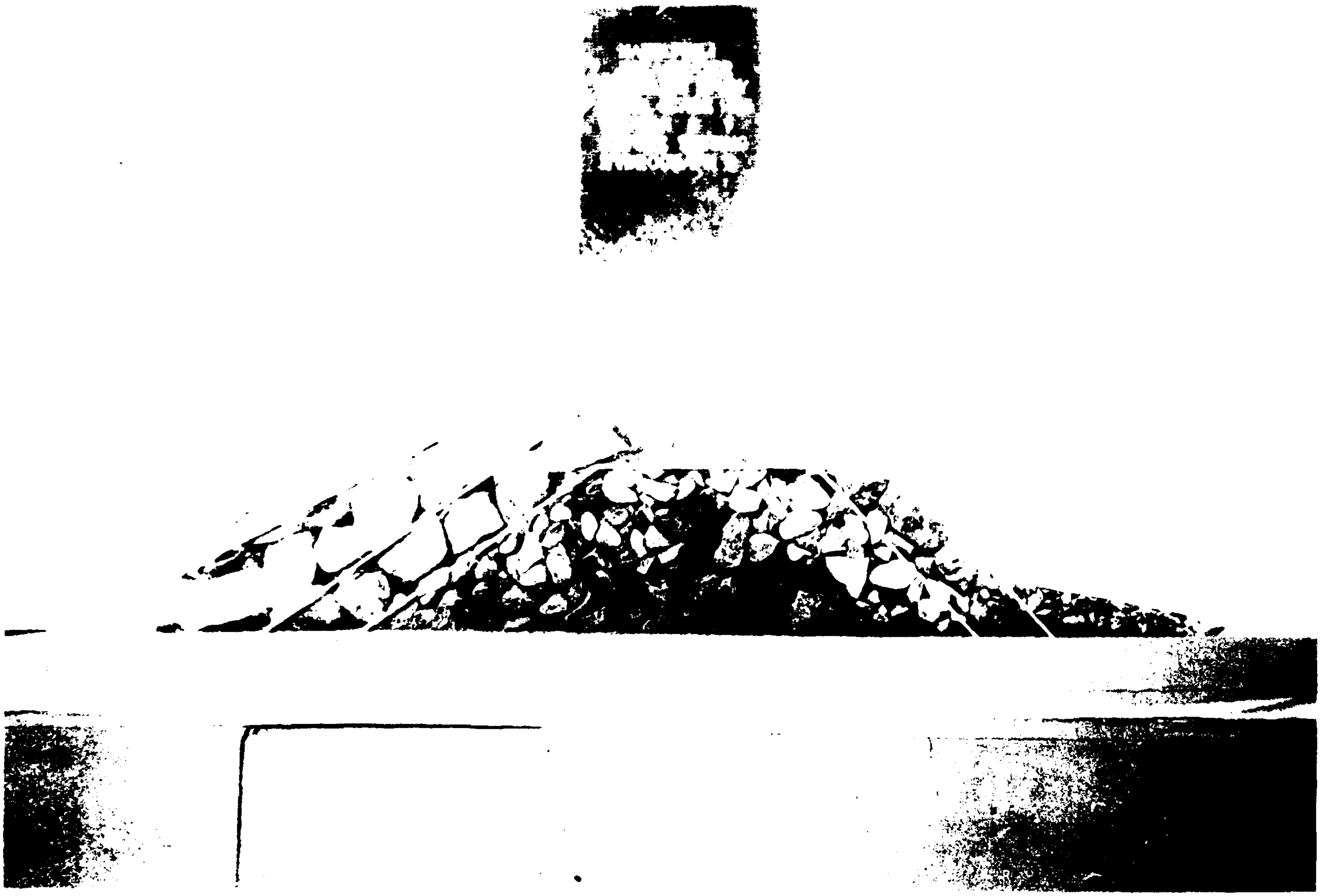


Photo 26. Side view of Section 3 before testing

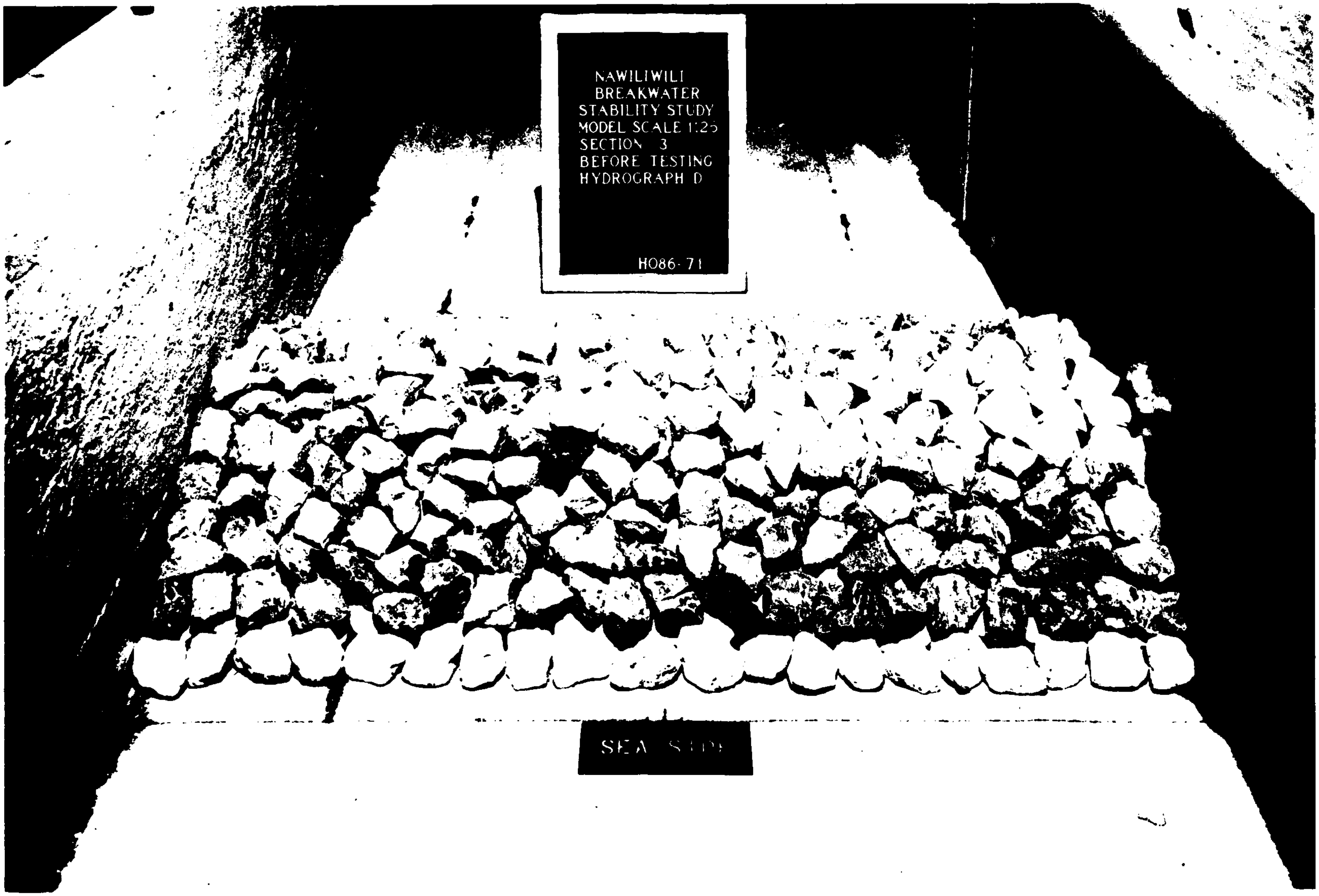


Photo 27. Sea-side view of Section 3 before testing

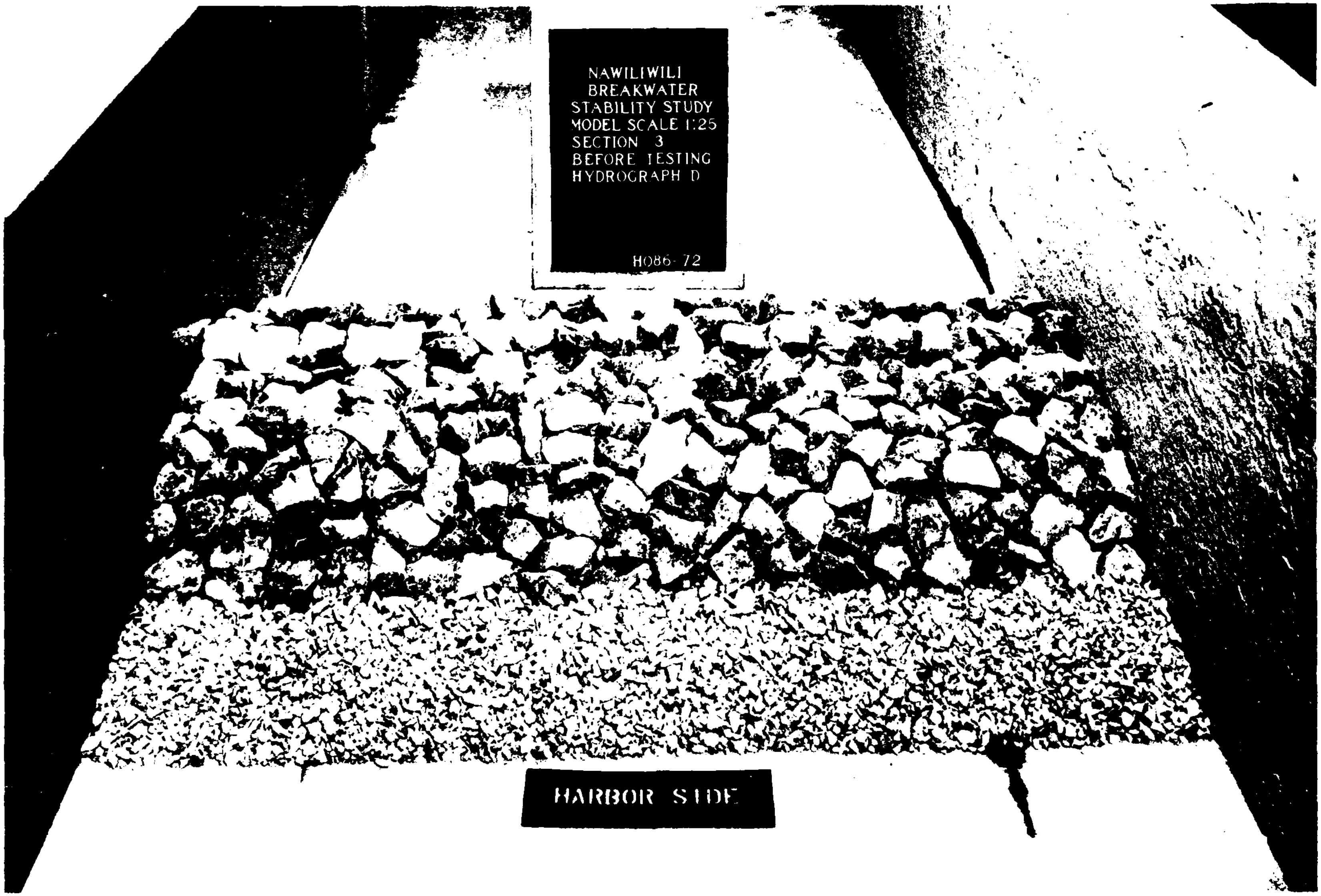


Photo 28. Harbor-side view of Section 3 before testing

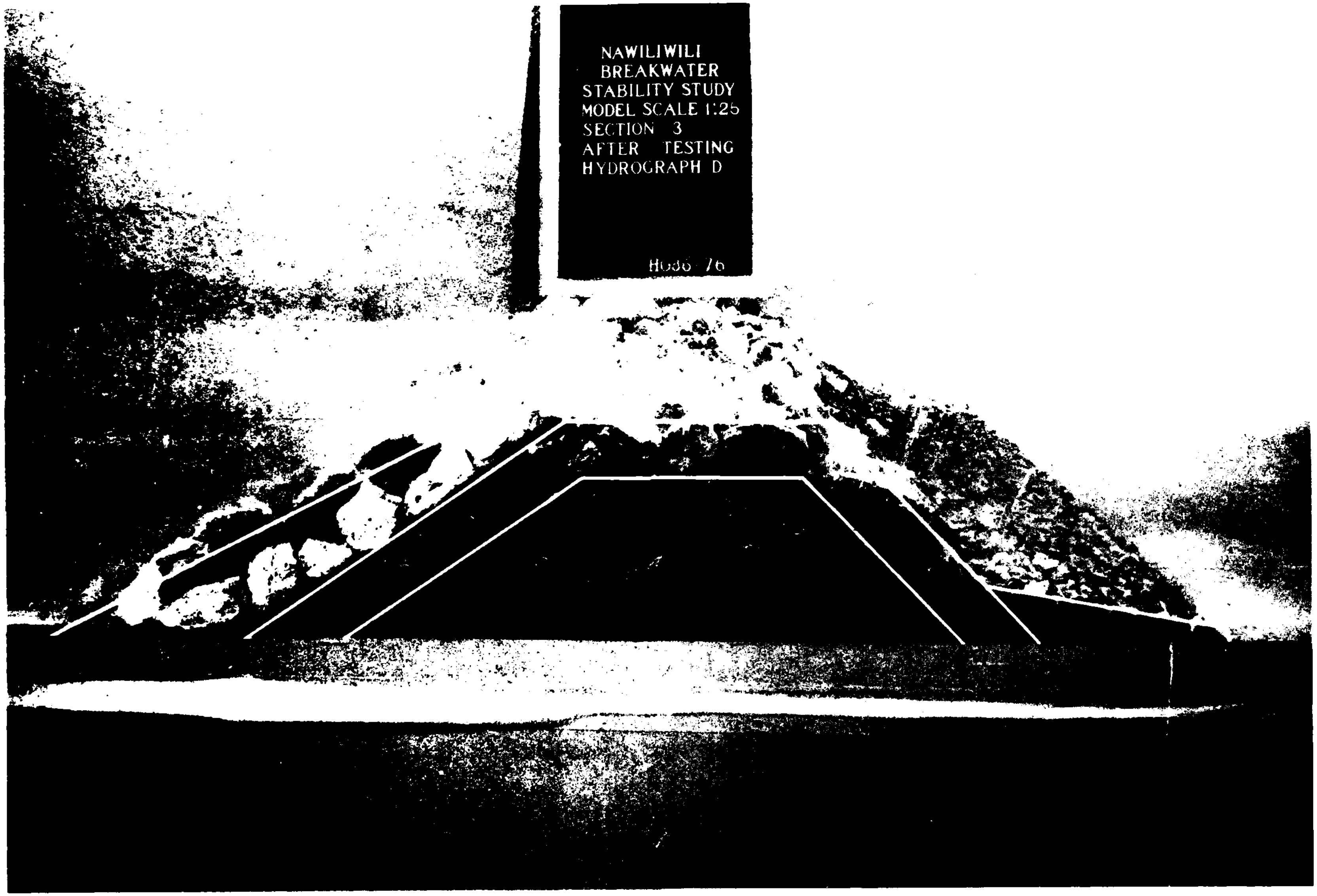


Photo 29. Side view of Section 3 after testing

Photo 30. Sea-side view of Section 3 after testing

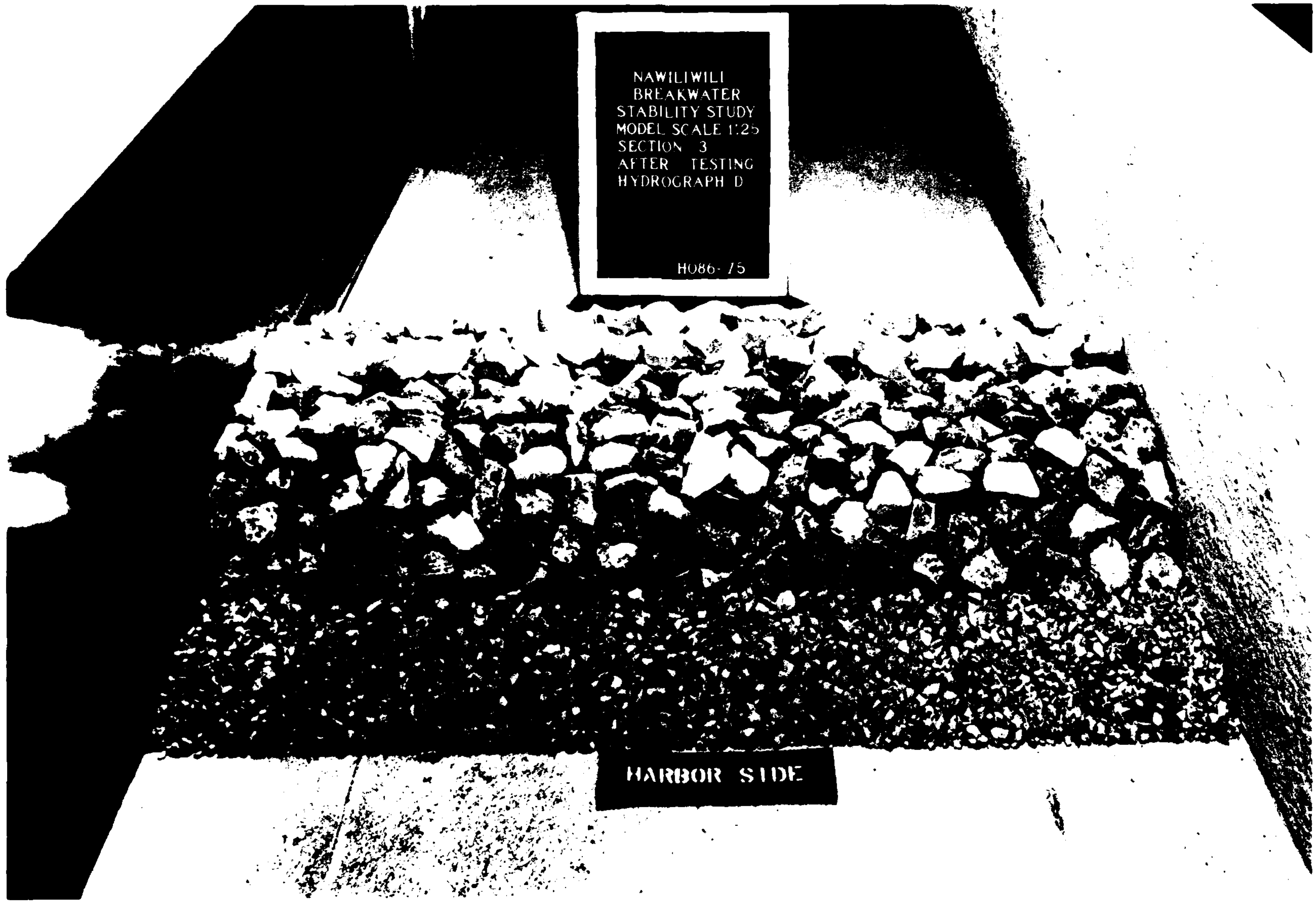
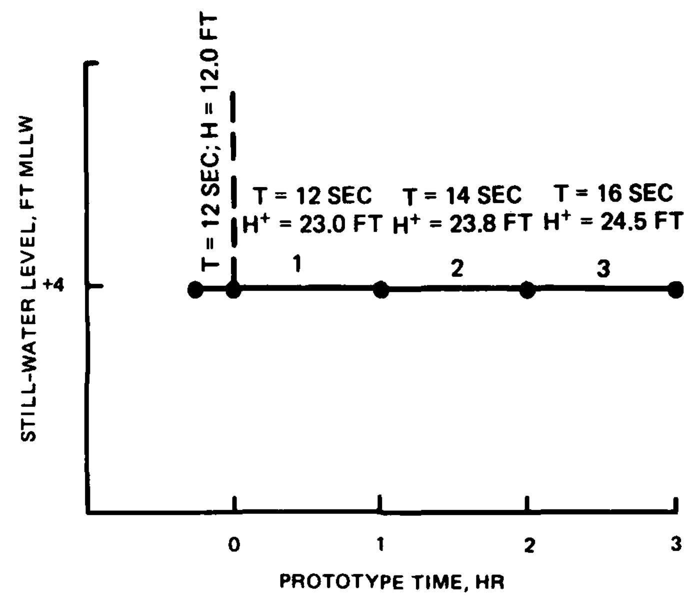


Photo 31. Harbor-side view of Section 3 after testing

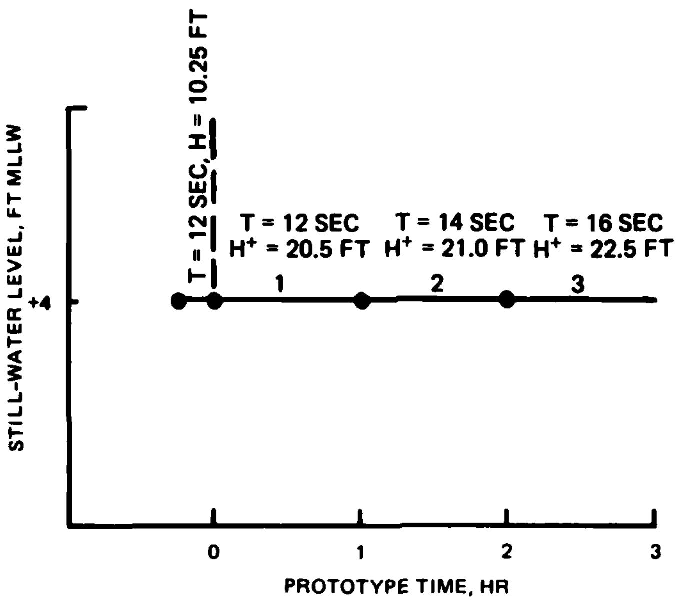


LEGEND

T WAVE PERIOD
H SHAKEDOWN WAVE HEIGHT*
H^+ WORST BREAKING WAVE HEIGHT*
1-3 HYDROGRAPH STEPS

* MEASURED AT THE -19.0 FT MLLW CONTOUR (TOP OF IV-ON-10H SLOPE)

HYDROGRAPH I
STA 19+50

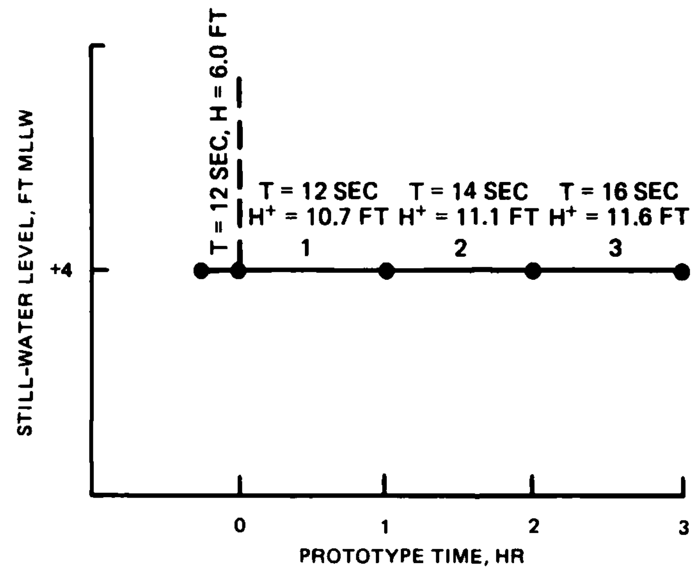


LEGEND

T	WAVE PERIOD
H	SHAKEDOWN WAVE HEIGHT*
H^+	WORST BREAKING WAVE HEIGHT*
1-3	HYDROGRAPH STEPS

* MEASURED AT THE -16.0 FT MLLW CONTOUR (TOP OF IV-ON-10H SLOPE)

HYDROGRAPH II
STA 14+00

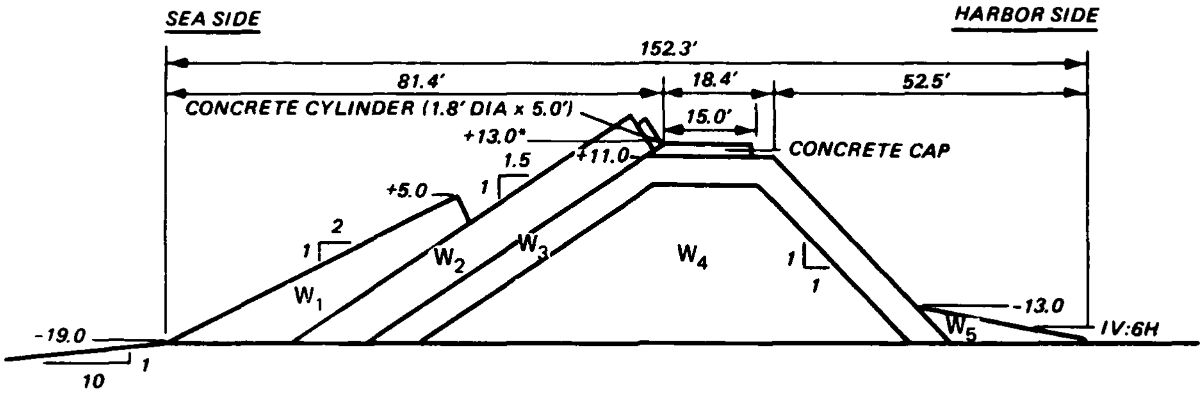


LEGEND

T	WAVE PERIOD
H	SHAKEDOWN WAVE HEIGHT*
H^+	WORST BREAKING WAVE HEIGHT*
1-3	HYDROGRAPH STEPS

*MEASURED AT THE -8.0 FT MLLW CONTOUR (TOP OF IV-ON-24H SLOPE)

HYDROGRAPH III
STA 10+00



MATERIAL CHARACTERISTICS

MODEL

W_1 = 0.861-LB DOLOSSE @ 138.5 PCF
W_2 = 1.125-LB TRIBARS @ 141.0 PCF
W_3 = 0.427-LB STONE @ 165 PCF
W_4 = 0.012-LB TO 0.047-LB STONE @ 165 PCF
W_5 = 0.001-LB TO 0.008-LB STONE @ 165 PCF

PROTOTYPE

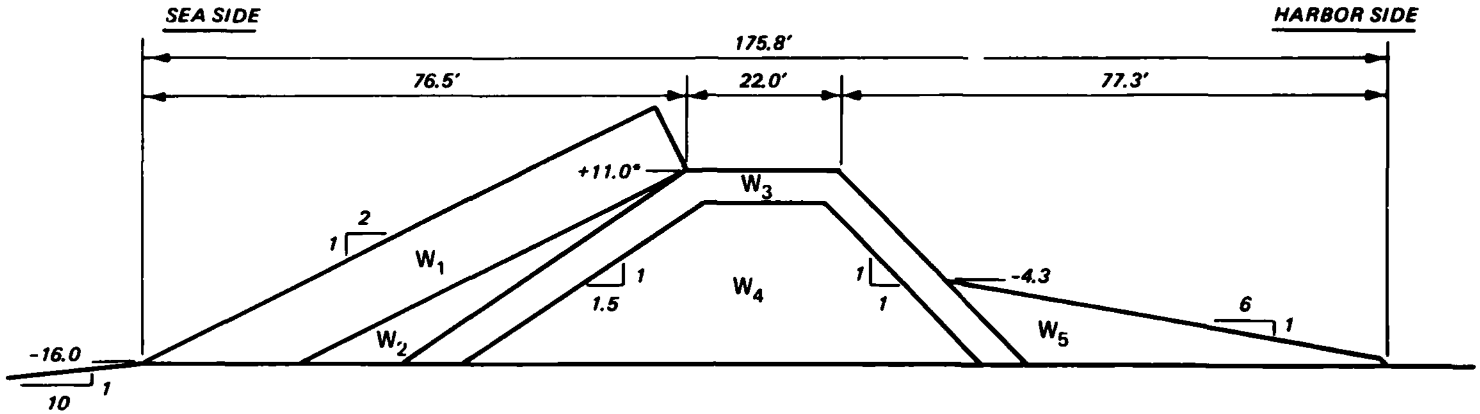
W_1 = 23,318-LB DOLOSSE @ 146 PCF
W_2 = 32,995-LB TRIBARS @ 146 PCF
W_3 = 18,000-LB STONE @ 156 PCF
W_4 = 500-LB TO 2,000-LB STONE @ 156 PCF
W_5 = 13-LB TO 350-LB STONE @ 156 PCF

MODEL AND PROTOTYPE PLACEMENT

W_1, ONE TO TWO RANDOM LAYERS
W_2, ONE UNIFORM LAYER
W_3, ONE KEYED AND FITTED LAYER
W_4, DUMPED CORE
W_5, DUMPED RUBBLE

*ELEVATIONS IN FEET REFERRED TO MEAN LOWER LOW WATER

SECTION 1
EXISTING CONDITIONS
STA 19+50

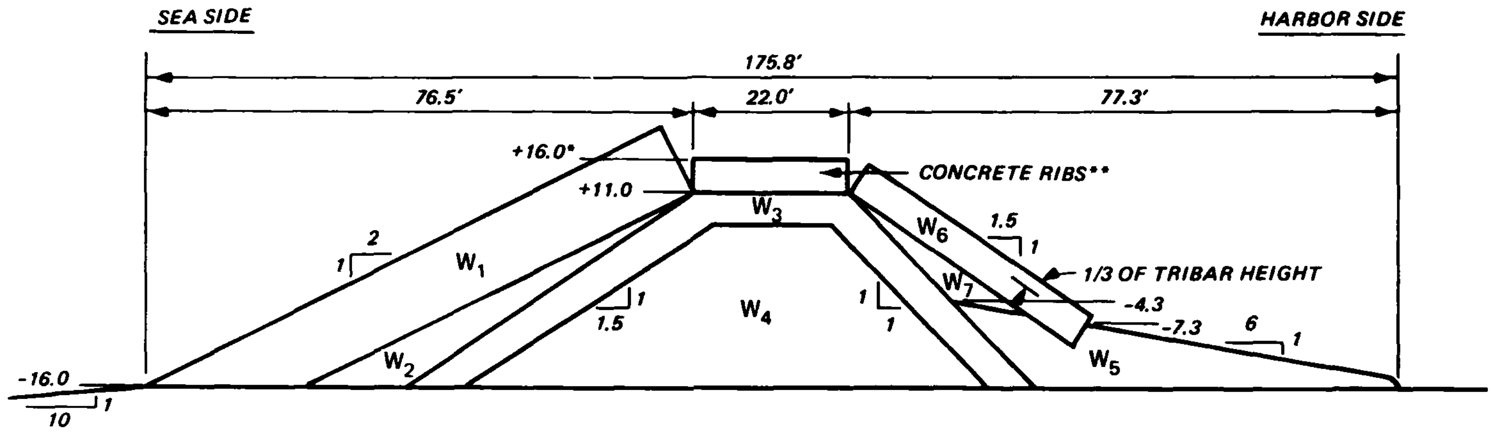


MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE	MODEL AND PROTOTYPE PLACEMENT
W_1 = 0.861-LB DOLOSSE @ 138.5 PCF	W_1 = 23,318-LB DOLOSSE @ 146 PCF	W_1, TWO RANDOM LAYERS
W_2 = 0.024-LB TO 0.095-LB STONE @ 165 PCF	W_2 = 1,000-LB TO 4,000-LB STONE @ 156 PCF	W_2, DUMPED FILL
W_3 = 0.427-LB STONE @ 165 PCF	W_3 = 18,000-LB STONE @ 156 PCF	W_3, ONE KEYED AND FITTED LAYER
W_4 = 0.012-LB TO 0.047-LB STONE @ 165 PCF	W_4 = 500-LB TO 2,000-LB STONE @ 156 PCF	W_4, DUMPED CORE
W_5 = 0.001-LB TO 0.008-LB STONE @ 165 PCF	W_5 = 13-LB TO 350-LB STONE @ 156 PCF	W_5, DUMPED RUBBLE

*ELEVATIONS IN FEET REFERRED TO MEAN LOWER LOW WATER

SECTION 2
EXISTING CONDITIONS
STA 14+00

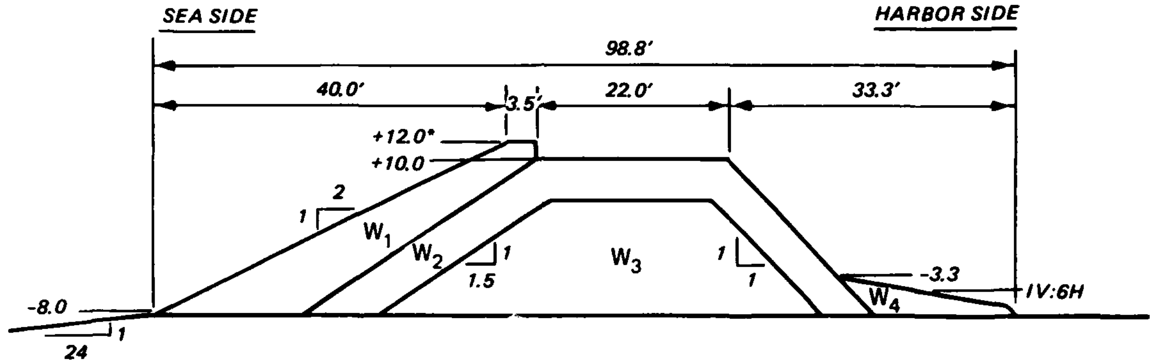


MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE	MODEL AND PROTOTYPE PLACEMENT
W_1 = 0.861-LB DOLOSSE @ 138.5 PCF	W_1 = 23,318-LB DOLOSSE @ 146 PCF	W_1, TWO RANDOM LAYERS
W_2 = 0.024-LB TO 0.095-LB STONE @ 165 PCF	W_2 = 1,000-LB TO 4,000-LB STONE @ 156 PCF	W_2, DUMPED FILL
W_3 = 0.427-LB STONE @ 165 PCF	W_3 = 18,000-LB STONE @ 156 PCF	W_3, ONE KEYED AND FITTED LAYER
W_4 = 0.012-LB TO 0.047-LB STONE @ 165 PCF	W_4 = 500-LB TO 2,000-LB STONE @ 156 PCF	W_4, DUMPED CORE
W_5 = 0.001-LB TO 0.008-LB STONE @ 165 PCF	W_5 = 13-LB TO 350-LB STONE @ 156 PCF	W_5, DUMPED RUBBLE
**W_6 = 0.427-LB TRIBARS @ 141.7 PCF	W_6 = 12,827-LB TRIBARS @ 146 PCF	W_6, ONE UNIFORM LAYER WITH TOE TRENCHED
**W_7 = 0.024-LB TO 0.071-LB STONE @ 165 PCF	W_7 = 1,000-LB TO 3,000-LB STONE @ 156 PCF	W_7, ONE TO TWO RANDOM LAYERS

*ELEVATIONS IN FEET REFERRED TO MEAN LOWER LOW WATER
**REHABILITATION MATERIALS

SECTION 2A
REHABILITATION DESIGN
STA 14+00



MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE	MODEL AND PROTOTYPE PLACEMENT
W_1 = 0.859-LB STONE @ 165 PCF	W_1 = 19,000-LB STONE @ 156 PCF	W_1, ONE TO TWO RANDOM LAYERS
W_2 = 0.814-LB STONE @ 165 PCF	W_2 = 18,000-LB STONE @ 156 PCF	W_2, ONE KEYED AND FITTED LAYER
W_3 = 0.023-LB TO 0.090-LB STONE @ 165 PCF	W_3 = 500-LB TO 2,000-LB STONE @ 156 PCF	W_3, DUMPED CORE
W_4 = 0.001-LB TO 0.016-LB STONE @ 165 PCF	W_4 = 13-LB TO 350-LB STONE @ 156 PCF	W_4, DUMPED RUBBLE

*ELEVATIONS IN FEET REFERRED TO MEAN LOWER LOW WATER

SECTION 3
EXISTING CONDITIONS
STA 10+00

APPENDIX A: NOTATION

A	Area, ft^2
H	Wave height, ft
K_Δ	Stability coefficient
L	Length, linear scale, ft
mllw	Mean lower low water
sta	Station, survey location where observations are taken
swl	Still-water level
S	Specific gravity
T	Wave period, time, sec
V	Volume, ft^3
W	Weight, lb
α	Angle of breakwater slope, measured from horizontal, deg
γ	Specific weight, pcf

Subscripts

a	Refers to armor units or stones
m	Refers to model quantities
p	Refers to prototype quantities
r	Refers to ratio of model quantities to prototype quantities (i.e., r = m/p)
w	Refers to water
1-7	Refers to different stone or armor unit sizes

END

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